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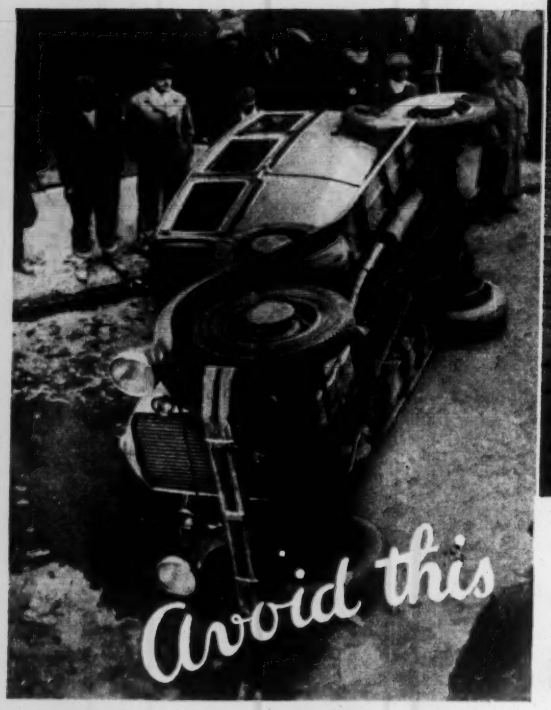

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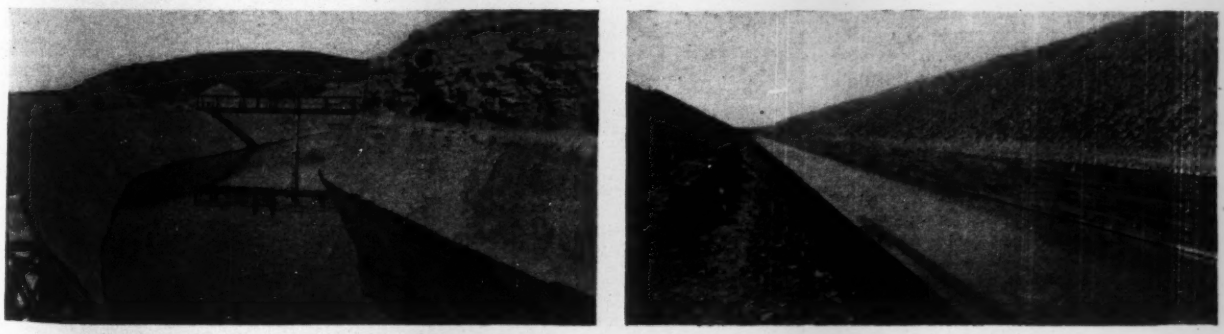
Irrigation and Power Combined by Texas Improvement District

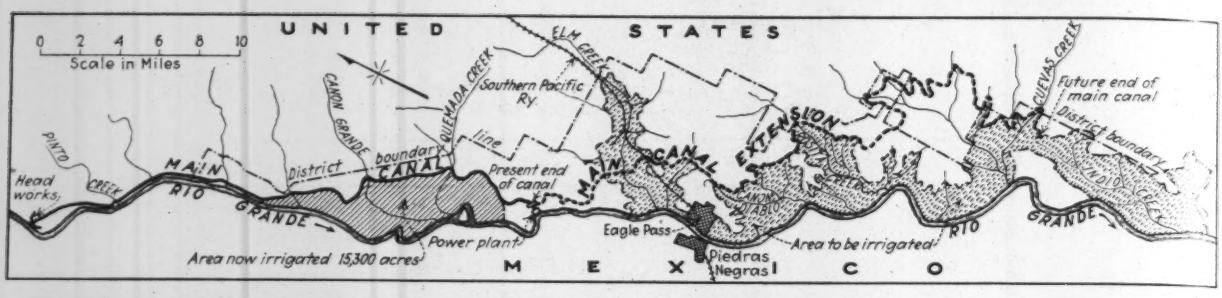
A COMBINED development of a section of the Rio Grande for both irrigation and power purposes, in which a hydro-electric station on the main irrigation canal is operated independently of the irrigation district, is now approaching completion near Eagle Pass, Tex. No diversion dam in the river is required, but the 32-mile canal with its gunite and redwood lining and inverted siphons under streams brought out many items of engineering interest in design and construction. The irrigation project and its construction methods are described in the following article. The hydro-electric development will be described subsequently. —EDITOR

WHILE soil conditions along the Rio Grande in southwestern Texas are favorable for agriculture, the rainfall is limited and uncertain, averaging only 20 in. per year. Irrigation with water pumped from the river has been applied successfully for some years, but only on a small scale. In 1926 the Maverick County Water Improvement District No. 1 was organized for the gravity irrigation of some fertile valleys in that county by means of a main canal 90 miles long. Of the 156,000 acres in this district, 60,000 acres can be irrigated by gravity and 25,000 acres by low-lift pumping. A diversion of 600 sec.-ft. from the Rio Grande was authorized in 1927 by the Texas state board of water engineers, which had approved a bond issue of $4,800,000 to carry on the project. An issue of bonds to that amount was then voted by the improvement district.

In 1928, however, a proposition was made by electric utility interests to add power development to the project, placing a hydro-electric station on the main canal about 32 miles below the intake and at an elevation of 82 ft. above the river, to which the tailwater would be discharged. This development was approved by the Federal Power Commission, and in October, 1928, the state board of water engineers permitted a total diversion of 1,500 sec.-ft. for irrigation and power, with a yearly diversion for irrigation of 150,000 acre-ft., or 2½ acre-ft. per acre on 60,000 acres. The district was reorganized in 1929 as the Maverick County Water Control and Improvement District No. 1. A second bond issue of $1,800,000 was voted, and a contract was made with the Central Power & Light Co., of San Antonio, to build a power plant of 13,500 hp. as an addition to its system. Besides the 60,000 acres to be irrigated by gravity flow from the canal under the present permit, it is possible to irrigate 25,000 acres of higher land by low-lift pumping, but no application has been made to the board of water engineers for a permit covering this additional area.



Either gunite or redwood lumber is used for the lined portions of the main canal.



The Rio Grande Canal at present is 32 miles long. Ultimately it will have a length of about 90 miles.

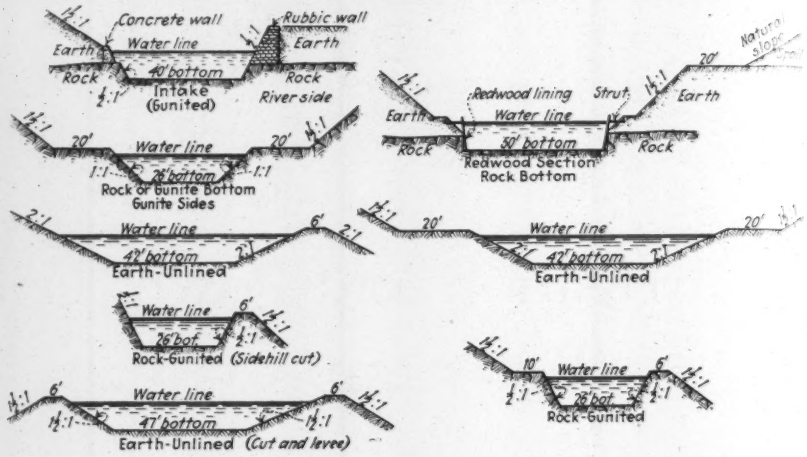
With a total authorized bond issue of $6,600,000, the bonded indebtedness up to February, 1932, amounted to $5,000,000. A flat-rate tax applicable to the irrigable land only has been set at $2.50 per acre and a water charge of $1 for the first 15 acre-in. of water delivered, with 25c. for each additional 3 acre-in. At present the 32-mile main canal and the power station have been completed, together with some 25 miles of lateral canals for irrigating about 15,000 acres of valley land in a tract 11 miles long and 3 to 5 miles wide, lying between the main canal and the river. Extension of this main canal for the 60 miles beyond the power house has not yet been undertaken.

Irrigation and power canal

Below the headworks the canal is partly in earth cuts, having a maximum depth of 55 ft., and partly in side-hill rock cuts. For its entire length it is within 2 miles of the river. At its lower end, some 10 miles north of Eagle Pass, the canal has attained a head of about 83 ft. above the low-water level of the river, and here the power plant is located.

About 80 per cent of the canal is in earth and was excavated by dragline machines, the spoil being used mainly to build banks or levees along the channel. For the earth section the bottom width is 42 to 47 ft., with side slopes of 1 (vertical) on 2 and 1 on 1½. In the long side-hill rock cut the bottom width is 26 ft., with side slopes of 1 on 1 and 2 on 1. With a normal 10-ft. depth of water, the freeboard varies from 1½ to 2 ft.

In lined sections the grade is 0.0002, and in unlined sections 0.0001 ft. per foot. Values of n in Kutter's formula for flow, confirmed in a recent report by Fred C. Scobey on the various sections of the canal, are taken at 0.018 for rock bottom and sides lined with redwood, at 0.0175 for rock bottom and gunite-lined sides, 0.015 for a completely gunited channel and 0.0225 for unlined earth sections. With these grades and coefficients a maximum flow of 1,600 sec.-ft. is obtainable, but right of diversion is limited to 1,500 sec.-ft. by the Texas state board of water engineers.



The sections used in the main canal were varied to suit changes in earth and rock formation.

About 6 miles of the canal has a gunite lining, the placing of which is described later. In addition, California redwood was used for lining a length of 6,600 ft. of rock cut immediately below the headworks, in order to expedite the work where seepage was particularly troublesome. Here the canal has steep side slopes of 6 on 1 and a bottom width of 50 ft. in unlined blue shale and limestone, dressed to reasonable smoothness. Assuming the value of n at 0.0075 for the sides and 0.02 for the bottom, or a mean of 0.018, with a slope of 0.0001 and a 10-ft. depth of water, this wood-lined section is estimated to have a discharge capacity of 1,780 sec.-ft.

The intake channel taps a natural pool in the river, making a diversion dam unnecessary. The canal headworks are at the right, and at the left is a sluiceway for return of excess flow to the river.

Canal structures and gates

The 32-mile canal, serving both irrigation and power purposes, has its intake about 42 miles upstream from Eagle Pass, the county seat. It taps a natural rock basin in the Rio Grande, formed by a riffle or reef, so that no diversion dam in the river is needed. The intake is a rock cut 2,500 ft. long, parallel with the river and having a 41-ft. width at the bottom, which is 10 ft. below the low-water level of the river. A rock retaining wall and spoil bank separate this channel from the river, while a low concrete retaining

wall is built on the land side. The average depth of rock cut in the intake channel was 6 ft.

At the lower end of the channel a dike about 48 ft. high separates it from the main canal, which here turns at about 45 deg. from the river. This dike, which excludes the river at high-water stages, carries a highway and small irrigation ditch. Through it pass three concrete conduits 160 ft. long, of horseshoe section, 13 ft. 3⅜ in. high and 17 ft. 6 in. wide at the floor. Grooves for stoplogs are provided at the inlet end, 42 ft. beyond which is the concrete well or gate chamber with a manually operated control gate of the roller type in each conduit. The gate openings are 10 ft. 2 in. high and 16 ft. wide. Silt collecting above the headworks may be washed out through a sluiceway to the river. The flow into this sluice channel is controlled by four manually operated 4x6-ft. gates, of the non-rising stem type, whose sills are 3 ft. below those of the head gates.

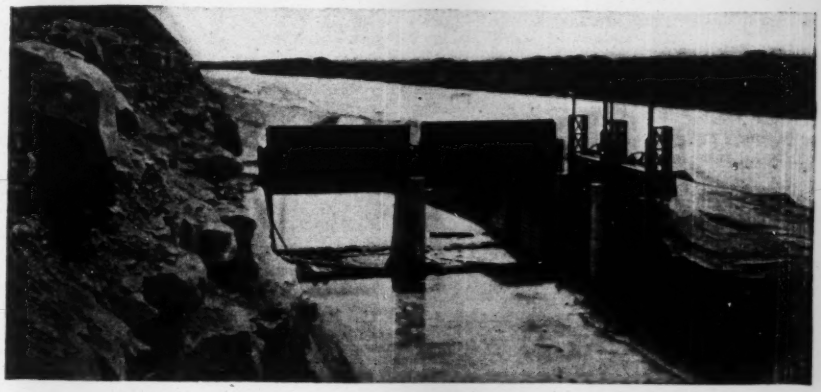
Intersecting streams are crossed by inverted siphons of reinforced concrete, each consisting of three parallel concrete conduits 10½ ft. in diameter. A 36-in. blow-off gate is provided for cleaning each siphon barrel. There are seven of these siphons, 100 to 500 ft. long. Box and pipe culverts under the canal carry the smaller drainage courses. Checks and spillways are located at three points, including the structure at the 32-mile point for the control of flow into the branch canal serving the power plant.

Canal excavating operations

Ideal weather conditions made possible a twelve-month continuous construction program for the Maverick County irrigation and power canal. The only part of the construction work governed by seasonal changes was the intake channel, where fall and spring rises in the river were to be expected. Although earth and sandbag cofferdams were placed in both the intake and sluiceway channels, rises on Oct. 6, 1930, and April 29, 1931, topped all protections and flooded the channels and headworks. However, little delay was caused by the flood or the subsequent clearing of the work.

Main canal excavation of 7,525,800 cu.yd. was started May 12, 1930, and completed Feb. 29, 1932. Four large diesel-driven walking draglines, five smaller crawler-type gasoline draglines and three gasoline shovels were used in this work. The largest of the diesel machines, having a 6-yd. bucket, 240-hp. engine and 120-ft. boom, moved 3,017,000 cu.yd., or a monthly average of more than 165,000 cu.yd., with 247,000 cu.yd. for its maximum month. The performance of other machines is given in the accompanying table.

Boggy ground encountered in several sections adjacent to the river necessitated the use of pontoons or timber mats for the support of the large draglines. Difficulty in handling this material was overcome by making repeated trips over the wet sections with the machines, excavating a central drain channel for successive lowering of the water table and the resulting drying of the material to a satisfactory handling condition. Most of the dragline material was dry, however, and in cuts suited to the machine capacities very little rehandling was necessary even in the heavy cuts. The maximum 55-ft. cut was drilled and shot with light powder charges for loosening a gravelly and semi-shale material, which was then readily handled but could not be penetrated by dragline buckets without being shot.



There are two check gates in the canal. Above the check gates are sluice gates discharging into the river.

Inverted siphons were used at the stream crossings.

Rock in bottom of earth cuts

Removal of rock in the bottom of some of the earth cuts, which was necessarily shot after the completion of earth excavation, presented a problem of disposal. It was solved by widening the earth cut to make room for rock spoil on the sides by a dragline working on the bottom. This is considered the outstanding feature of dragline performance on the job. While the wear on buckets of the draglines was severe, it was more than compensated for by the saving in speed and ease of disposing of rock in the bottom of heavy earth cuts. Rock that was well shot was easily handled by standard toothed buckets.

Gelatine and ammonium dynamites were used in blasting some 150,000 cu.yd. of bottom-cut rock. In some cases face shooting proved most advantageous, and in others the material was shot in the rough in one or two lifts, the second being then trued by means of closely spaced air-drill trim shots pulled after removal of first-shot material. Air-drill trimming was used on the rock in the intake channel and in the section immediately below the headworks.

The most difficult rock work occurred in a side-hill cut about 3½ miles long. Holes sprung by dynamite were loaded with black powder, the loosened material being excavated and cast by gasoline shovels. Two to three lifts were required to bring the cuts to grade. Considerable rehandling of material was required in this work, and quite often two shovels were worked in conjunction, one hoisting material from the cut to a bench and the other casting it on the spoil bank. Portable compressors supplied the air for the jackhammer drills, except that an electrically driven compressor was used below the headworks. Fills intervening between the side-hill rock cuts were made with small gasoline draglines and were puddled to reduce settlement. Rock work amounted to about 685,000 cu.yd.

To prevent leakage, a 1½-in. gunite

PERFORMANCE OF CANAL EXCAVATORS

Bucket, Cu.Yd.	Hp.	Length of Boom, Ft.	Total Yardage, Cu.Yd.	Monthly Average, Cu.Yd.	Best Month Cu.Yd
4	120	76	*1,286,000	61,200	118,000
4	120	67	1,597,000	83,000	113,000
4–5	180	80–120	1,010,000	76,000	111,300
1½	125	40	142,000	30,600	46,150

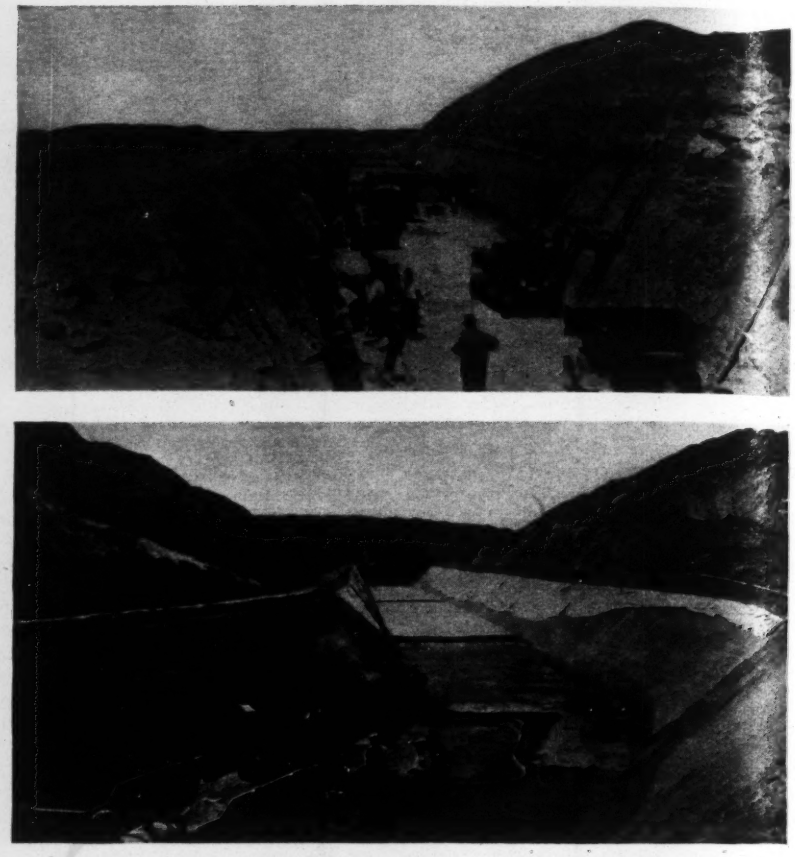
*Includes about 150,000 yd. of rock.

lining was placed in all side-hill rock cuts, about 4¼ miles, and in 1¾ miles through pervious material such as sand, gravel and loose rock. A 2-in. gunite lining was used at the inlet and outlet ends of siphons, checks and spillways. In addition, about 41,000 sq.ft. of ½-in. non-reinforced gunite was applied on masonry walls and on exposed rock in order to prevent air slacking, while 35,000 sq.ft. of stone at the mouth of the sluiceway channel was bonded by gunite averaging 3 in. in thickness.

For the canal the lining extends over the bottom and up the side slopes to 18 in. above the water line. It has a minimum thickness of 1½ in. and is reinforced by wire netting of 4-in. mesh pinned to the slopes and about ½ in. from the ground surface. In soft material the excavation was dressed to a smooth slope before being lined. In rock considerable difficulty was encountered in building up the surfaces of over-shot sections. Cavities were filled with rock set in clay mortar, or in cement mortar where clay was not available. Minor rock projections were broken off with air-operated bull points. Where seepage occurred on rock surfaces that were lined, weepers or drain pipes were set to prevent the development of hydrostatic pressure behind the lining.

The 1:4 cement-sand mixture was prepared in revolving-drum mixers of one-sack capacity, having a revolving screen and discharger attached for the purpose of screening the dry mixture as it flowed by gravity to a chute leading to a material bin. Motor trucks then hauled this dry mix to the cement guns. Three of these central mixer plants were used, located near the middle of the sections being lined. For the grouting there were three outfits, moved from place to place as the work progressed. Each was equipped with a material truck and trailer, a cement gun, portable compressor, tractor, water tanks, a mixing plant, pumps and auxiliary equipment. The gun was mounted on a four-wheel trailer, the material truck backing up and dumping the dry mix on the bed of the trailer, from which it was shoveled by hand into the gun. The outfit moved when all lining within reach of 100-ft. hose was completed. In long cuts two outfits were used, working from each end toward the middle.

Water for wetting the graded surfaces and for shooting and curing the gunite was obtained from the Rio Grande. High-pressure pumps on the river bank delivered the water through a maximum length of 7,000 ft. of 2-in. pipe to the center of each section to be lined. From this point a 2-in. pipe was laid along the canal, with outlet connections 100 ft. apart. For curing, the lining was kept sprinkled with water for at least seven days by means of hose attached to the 2-in. pipe. Although no expansion or construction joints were formed in the lining, there were only a few temperature cracks, with some larger cracks where the lining had insufficient support.



Lining the canal with gunite. Men in foreground of upper view are building up a surface made rough by overbreak. In the foreground of the lower view the dry-mixed materials are being delivered to the cement gun. Burlap was used to protect the fresh concrete during the curing stage.

Special methods were required for lining 1¾ miles of cut in bank sand and gravel on a section below the ground-water level, where water 2 or 3 ft. deep was flowing in the canal. Here it was necessary to isolate 200-ft. lengths by bulkheads and to handle the flowing water by centrifugal pumps. Owing to the continual infiltration of water at the bottom and sides, it was necessary to construct rock drains and to install weeper pipes closely spaced, in order to provide a fairly dry surface on which to place the lining.

Each cement gun could place about 5,000 sq.ft. of lining per day. The total area on this work was about 2,000,000 sq.ft. placed, with a monthly average and maximum of 160,800 and 277,440 sq.ft., respectively.

Redwood lining

For the 6,600 ft. of rock cut lined with California redwood, as already noted, posts 6x6 in., 12 ft. high and space 4 ft. c. to c., were secured at the bottom by ⅝-in. galvanized U-bolts grouted into the rock, while at the top they were held by 2x4-in. redwood deadmen buried in trenches. These posts support a 1½x8-in. cap and 1½x8-in. sheeting fastened with 20-penny copper-weld spikes. The sides are steep, inclined 6 on 1.

Concrete structures

The concrete work on the canal was widely scattered, but its major yardage was at the headworks and the seven triple-tube inverted siphons. Minor yardage occurred in box and pipe culverts, bridges, check and control structures and lateral outlets. The principal problem involved in constructing these minor structures was that of distributing materials and supplying water, tank trucks being used to haul water in many instances.

Lateral canals

A lateral canal system of about 25 miles for distributing water from the main canal to the 15,000 acres to be irrigated as the first part of the project was completed Jan. 31, 1932, requiring 244,000 cu.yd. of excavation and 730 cu.yd. of concrete. Excavation was done by means of teams and fresnos and in some of the larger sections by a 1½-yd. crawler dragline.

The irrigation phase of this project

was designed by W. L. Rockwell, San Antonio, Tex., as supervising engineer for the water control district, with H. A. Beckwith as assistant engineer and K. K. Prestridge as engineer of design. The general contractors were Ulen & Co., Lebanon, Ind., with T. L. Jacobi as construction engineer. Among the principal subcontractors were: the Trinityfarm Construction Co., Dallas, Tex.; Bart Moore, Inc., San Antonio; Barker Brothers and W. H. Davison, Amarillo, Tex., and Cushman, Ark.; and the Gunite Concrete & Construction Co., Kansas City, Mo. The hydro-electric development, which will be described in a later article, is a separate and independent project, the district selling a certain quantity of water to the Central Power & Light Co., San Antonio, Tex., for which company L. F. Harza, Chicago, is consulting and designing engineer. The Maverick County Water Control and Improvement District No. 1 is governed by a board of five members; its manager is T. A. Caldwell, who was job manager for the general contractors.

Simplified Lane Marking on Massachusetts Roads

New standards virtually limit traffic lines on road surface to curves and crossings equipped with signals

NEW STANDARDS for traffic-lane marking adopted by the Massachusetts highway officials for state roads confine lines on the pavement to blind curves, tops of hills and intersections governed by traffic signals. Virtually all other sections of road are left unmarked except under special conditions. Where markings are allowed they are limited to simple lines; no lettering or symbols are allowed on the pavement, but instead, if warning or instructions are anywhere deemed necessary, roadside signs are provided.

The adoption of these few and simple markings is based on the conviction from experience that the general use of lane lines confuses the motorist, often delays him unnecessarily and tends to breed in him a disregard of the markings where hazard really calls for their observance. In the new standards, therefore, traffic lanes are marked only where hazard is involved in departing from fixed lines of vehicle movement. In brief, there are no lines at all on four-lane roads except on demonstration of necessity, there are no lines at all on dual-surface three-lane roads, and there are no lines at intersections (railway or highway) where there are no signals. The only lines on any other roads are at signaled intersections and on curves (vertical and horizontal) where the clear sight distance is 300 ft. or less. The accompanying diagrams show the lane markings at intersections and curves.

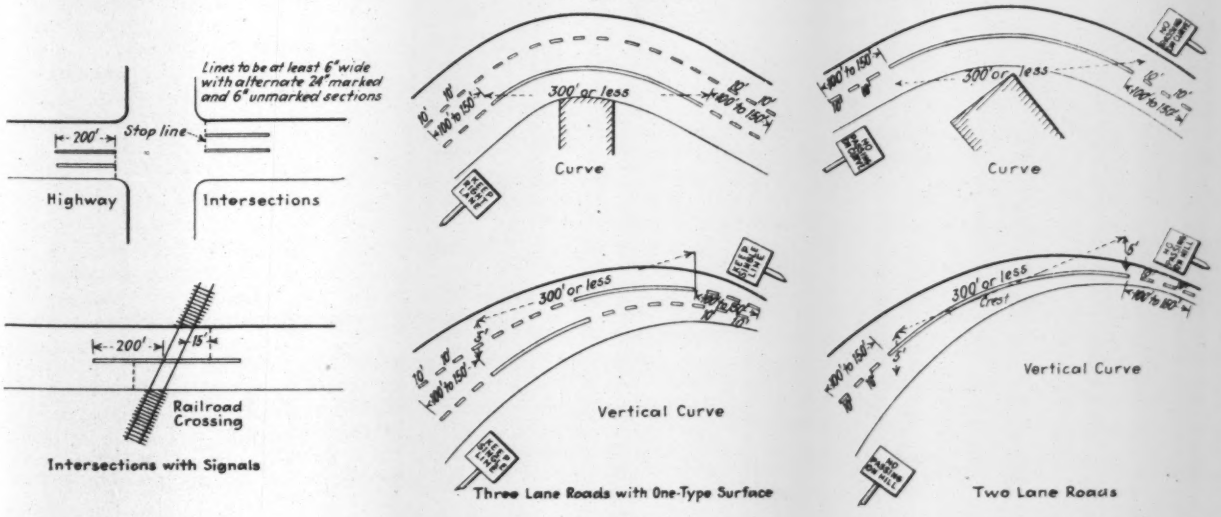
It is stated that no white lines are to be painted on four-lane surfaces except on approval. This approval in some cases is granted, as, for instance, on roads leading to beach resorts where particularly heavy traffic is at times experienced.

The development of the standards governing the use of lines on three-lane one-type surfaces is interesting. Previously, whenever it was thought necessary to insert some control on a horizontal or vertical curve, a white line was painted in the center of the surface, thus changing a three-lane road to a two-lane surface. The effect on the motorist of the frequent transition from a three-lane road to a two-lane surface and back again was found to be unpleasant and frequently prevented him from passing a slow vehicle ahead while going down hill or coming out of a curve, even though there were no oncoming vehicles to make such a passage unsafe. The present standards confine motorists to a single lane until they reach the top of the hill, at which point they are in a position to see what may be ahead, and are as a result given two lanes so that they may pass if conditions are otherwise safe. Similarly on horizontal curves the motorist on the inside is confined to one lane, reserving the two outer lanes for passage in the opposite direction, in which case advance warning of such traffic lines and conditions is afforded by appropriate signs.

The highway department is not yet convinced that vehicles should be prevented from passing while going uphill, because much of the delay experienced by motorists is caused by heavily loaded trucks going uphill. These same trucks go much faster downhill, so that it is possible, if passing is to be prohibited, that it should be while going downhill, as there would be less inconvenience and loss of time imposed on the motorist. It is intended to have the traffic engineers of the department study all these phases during the present summer.

In every case deviation from any of the standards is permitted on approval. For instance, fog is particularly prevalent on some sections of state highway, and where this is so, white lines will be painted on the outer edge of the surfaces of the sections involved. The new standards have been worked out by the engineers of the department of public works, of which A. W. Dean is chief engineer.

Standard traffic-lane markings adopted for state highways by the Massachusetts department of public works.



Three Articles Giving New Data on Soil Mechanics

I — Soil-Loading Test Practice in Europe

By Karl Terzaghi
Professor, Technische Hochschule, Vienna, Austria

THE PAST FEW YEARS have witnessed widespread acceptance of the importance of soil mechanics. The practical significance of the various theories developed in this relatively new field of engineering science is gaining appreciation from their actual use on the job. Three articles reflecting current progress and recounting recent developments are presented here. In the first article Dr. Terzaghi, international authority on soil physics, whose work is familiar to American engineers, describes and appraises the unusually ingenious apparatus and methods devised by European engineers for determining the ability of soils to carry foundation loads.

In the second article are described tests conducted at the Massachusetts Institute of Technology during the past few years, which show an increase of compressibility of remolded clay compared with the same clay in the undisturbed state. The relation of this determination to the design of foundations in clay is discussed in the article that is condensed from the paper by Mr. Casagrande read before the Boston Society of Civil Engineers.

A practical application of the principles of soil mechanics in the United States is illustrated in the third article, by Stephenson and Feingold, of the University of Michigan.

Taken collectively, the three articles constitute an appreciable contribution to the available information on soil mechanics. —EDITOR.

IN SPITE OF the difficulties connected with the interpretation of the test results, loading tests are still considered by many engineers and building departments in Europe as the only fairly reliable method for investigating the bearing capacity of the soil. Hence from time to time new devices appear on the market to perform the loading tests relatively cheaply and rapidly. Some of the tests that have been advanced are listed here briefly, and references are given to sources of more complete information.

Invention active

In Germany a device patented by Buchheim and Heister in 1916 is still used. Since the leverage is very great (1:140), a small load is sufficient to produce pressures as great as 5 tons. The areas on which the pressure acts are 0.3, 0.6 and 1.7 sq.ft. The device of Stern, which was accepted in 1927 by the building authorities of Vienna (penetration test with conical body of 3-in. diameter), has practically disappeared. It is replaced by an apparatus patented in 1930 by Fischer. (K. Fischer, "Ergebnisse von Baugrundprüfungen," *Ztschr.d.öst. Ing. u. Arch.Ver.*, 1930.) The pressure is exerted by a hydraulic jack that acts on an area of 0.22 sq.ft.

A rapidly increasing tendency is noticed to replace the traditional surface loading tests by tests that are performed in drill holes at different levels below the surface of the ground. A primitive predecessor of the modern types was used by the Swedish geotechnical commission in connection with its investigations made during the years 1914-1922. By determining the force required to press an auger-shaped tool into the ground the data were obtained for plotting a resistance diagram of the soil (resistance against depth).

Several years ago, in connection with the preliminary work for the construction of the bridge of Ligingo near Stockholm, John Olsson, of the Swedish geotechnical commission, used a device for estimating the shearing strength of the soil (very soft clay) at different depths below the bottom of the bay. (Described in W. Krantz, "Geologie im Ingenieurbaufach," 1927.) For the purpose, he used steel plates with a height of 20 in. and a width of 12 in., which were forced into the ground in a vertical position. The shearing resistance of the soil was computed from the torque required to turn the plates around their vertical axis.

In 1929 Wolfholz took a patent on a device for making loading tests on the bottom of a drill hole lined with a casing of 14-in. diameter. (*Siemens Ztschr.*, Bd. 10, N. 2, February, 1930.) The pressure is exerted by means of a hydraulic jack and acts on a circular slab with a diameter of 12 in. The patent was taken over by the Siemens Bau Union in Berlin and was used for the first time on a large scale in connection with the foundation for the electric-power plant "West" in Germany. Later Dr. Kögler in Freiberg secured a patent for another device for making loading tests in drill holes. In his device the pressure is exerted in a horizontal direction against the walls of the drill hole. The pressure is measured by means of an oil-pressure manometer. At the meeting of the "Deutsche Gesellschaft fuer Bodenmechanik" (December, 1930) Professor Hertwig presented a preliminary report on a device for the determination of the average elastic constants of the subsoil from seismographic records. The vibrations of the ground are produced by means of an electromotor operating a tamping device that exerts alternating blows on the surface of the ground (dynamic loading tests).

Interpretation lacking

The preceding summary indicates intense activities directed toward the construction of efficient tools for making static or dynamic loading tests. Little progress will result from these efforts unless we succeed in establishing fairly reliable methods for the interpretation of the test results. No such rules can be secured except by close cooperation between sound theory, laboratory investigation and systematic observation on full-sized structures. For the time being the situation is rather chaotic.

Rules for loading tests

For interpretation of the results of the traditional loading test performed on the surface or on the bottom of open pits, one finds in European literature no less than three different rules:

1. The admissible soil pressure is equal to one half of the ultimate bearing capacity.

2. The admissible soil pressure is equal to the limit of proportionality (rule for the interpretation of the results obtained with testing device of Fischer in Vienna).

3. The admissible soil pressure is equal to one half of the pressure required to produce a settlement of $\frac{3}{4}$ in. (German standard specifications).

All these prescriptions represent merely a makeshift attempt to escape the consequences of our ignorance of the relation that exists between the size of the loaded area and the settlement. Up to this time, theory has informed us merely on the extreme possibilities but

not on the intermediate cases. Since the inadequacy of the substitutes for complete knowledge is generally recognized, many investigators concentrate their efforts on the problem of bearing capacity.

The large-scale experimental investigations that were recently made along this line, by H. Hugi (Dissertation, "Untersuchungen über die Druckverteilung im örtlich belasteten Sand." Zürich, 1927) and E. Gerber (Dissertation, "Untersuchungen über die Druckverteilung im örtlich belasteten Sand." Zürich, 1927) and Kögler and Scheidig (F. Kögler und A. Scheidig, "Die Druckverteilung im Baugrund." *Die Bautechnik*, Freiberg, 1928) gave us conclusive information on the distribution of the stresses in the soil beneath loaded areas. Their results together with the results obtained by H. Press ("Baugrundbelastung mit Flächen verschiedener Grösse." *Die Bautechnik*, 1930) have also informed us, within the limits of the experiments, on the influence of the size of the loaded area on the amount of settlement. It remains to make the step from our knowledge of the distribution of the stresses to the computation of the corresponding strain. To make this step requires making definite assumptions concerning the elastic properties of the soil.

Facts and theories disagree

The theoretical investigations of M. van Iterson ("Résistance du Terrain à la Charge des Fondations," Soc. des Ing. Civ. de France, 1928) and of F. Schleicher ("Zur Theorie des Baugrundes." *Der Bauingenieur*, 1926) were based on Boussinesq's equation and on the assumption that the modulus of elasticity is everywhere the same, independent of depth. There is ample experimental evidence that this is by no means the case. Hence it is not surprising to find that the theoretical results fail to agree with the observed facts. The assumption on which the theory of Williams and Housel is based is still more arbitrary. This assumption is that the value of the coefficient of incompressibility is constant to a certain depth H, and equal to infinity below this depth. The Williams-Housel hypothesis has not found its way into European scientific literature.

Elastic behavior studied

In order to arrive at a satisfactory agreement between theory and practice we must adapt our assumptions to the laws of elastic behavior of soils in a state of part confinement. Unless these laws are thoroughly known, our assumptions inevitably remain arbitrary. To increase our knowledge of these laws, tests are being conducted in the laboratory of the writer in Vienna on soils under the influence of stresses that act both in a horizontal and in a vertical direction. The results of these investigations will guide our next steps.

Until we succeed in reconciling our knowledge of the distribution of stresses and the laws of elastic behavior with the known test results, there is no use attempting to compute the settlement of full-size structures from the results of loading tests. In the meantime, we are obliged to continue interpreting the results of loading tests on the basis of more or less arbitrary rules.

Loading tests of a special type are those that are performed in the laboratory on confined undisturbed drill samples for the purpose of securing data for the computation of settlements due to the gradual consolidation of soft strata. Investigations of that type are being made in the laboratory of the writer in Vienna, with the financial assistance of the foundation committee of the American Society of Civil Engineers, with drill samples from the ground underneath existing structures. The investigations have matured to a stage sufficiently advanced for practical application.

* * *

II—Compressibility of Clay Increased by Remolding

By Arthur Casagrande

U. S. Bureau of Public Roads, Massachusetts Institute of Technology, Cambridge, Mass.

A CLEAR understanding of stress, deformation and rupture, as applied to soils, is the first essential in the consideration of their ability to carry foundation loads. Theoretically the meaning is the same as in the case of the more rigid structural materials. Their relationship in soils is very different. Contrary to the case of most structural materials in which strength or resistance to rupture defines a safe bearing load, in soils it is the stress-strain relationship of the material as computed in deformation that determines an allowable bearing load.

Stated another way, in structural design the number of cases in which the load is limited by deformation is small due to the great rigidity of the structural materials. For soils the opposite is generally the case. Soil is so much more compressible, and its deformations are so much larger than those of the common construction materials that in the great majority of foundation problems the loading of the soil must be limited to a value that is much below the bearing load as determined by its shearing strength. Foundation design has then the purpose of determining the allowable load for an allowable soil deformation. This requires determination of the compressibility of soils. If the amount of compression that a soil will undergo under a load and the rate at which compression will proceed can be determined, the problem of how to compute, at least approximately, the settlement of a structure will have been solved.

Compressibility of clay

The consideration of foundation soils in this article is confined to clay. Using the compression method developed by Dr. Karl Terzaghi, tests were made at the Massachusetts Institute of Technology of undisturbed and remolded clays. The clays were from two sources, the Laurentian Valley in Canada and localities around Boston. Typical results only are given. In each case a disk was cut from the undisturbed clay; the clay around the cut was then thoroughly remolded, and the two samples were tested in identical manner. Typical test results on the two Canadian clays are shown by Figs. 1 and 2.

The curves of the undisturbed and the remolded clay are to be observed in comparison. Apparently nature builds up such clay deposits in a peculiar structure that is so carefully arranged that it is many times stronger than the irregularly arranged grains in the remolded clays. Curves of the Boston clay tests show a similiar relationship of undisturbed and remolded clay pressure curves but with less difference in compressibility.

Physical difference explained

In order to explain the difference in the physical properties of clay in undisturbed and remolded states, it is assumed that the flocculent structure that is formed during the sedimentation process does not experience uniform consolidation in all its parts. On account of the exceedingly slow increase in weight of the deposit accumulating above, the pressure acts to bring closer the sand and silt particles and to compress the bond clay where they approach contact, but it leaves in the arch cavities a relatively uncompressed mass of clay. Imposed pressure acts along the same high-strength paths in undisturbed clay. Remolding destroys the bond of highly consolidated clay between the silt grains, diffuses the soft clay in the arch cavities and so destroys the original strength structure.

The driving of piles has a remolding action on the clay adjacent to the piles. Lacking experimental data, opinion as to the radius of the remolding effect is drawn from: (1) consideration of the character of the movement of clay which is displaced by the pile and which is pressed upward between the piles (Fig. 5): (2) the increase in driving resistance after the pile rests in the clay for

some time, which is due to part consolidation of the remolded clay surrounding the pile; and (3) observation of the vibration and the slight lateral flexure at each blow and the quaking and the jerky movement of the soil between piles in constructing a pile foundation in the vicinity of Boston. The estimate arrived at for piles of common dimensions is: the clay immediately surrounding the pile to a distance of one-half the diameter of the pile is completely remolded, and to a distance of

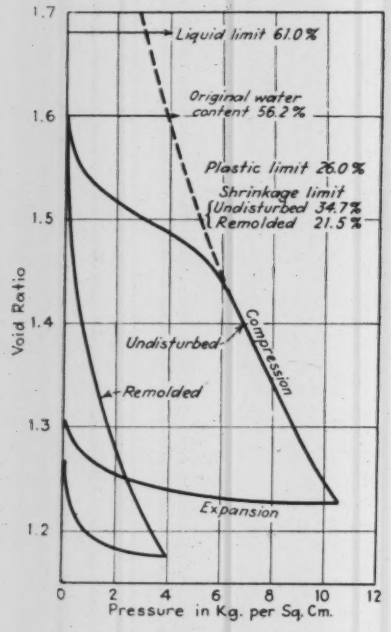


Fig. 1—Compression curves for a sea-deposited Laurentian clay tested in undisturbed and remolded state.

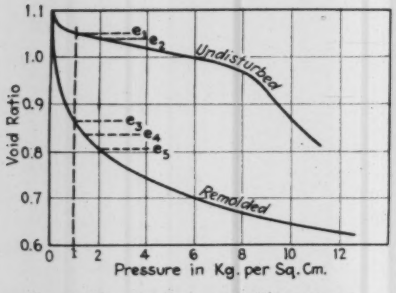


Fig. 2—Compression curves illustrating increased compressibility of Boston clay due to remolding, as by driving piles.

1½ diameters is sufficiently affected to result in a large increase in compressibility.

By remolding a large portion of the clay mass embraced by the pile assembly, the effect of piledriving is to increase the compressibility of the foundation area. While the gradual distribution of the load that rests on the piles into the soil tends to decrease the total amount of settlement, the increase due to remolding of the clay is greater by far. The major portion of this increase in settlement is not due to the load that

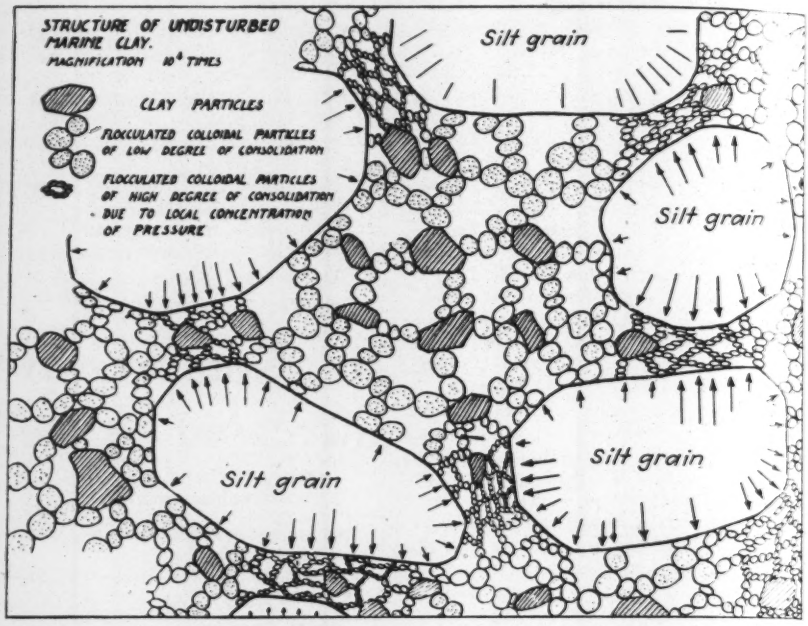

STRUCTURE OF UNDISTURBED MARINE CLAY.
MAGNIFICATION 10⁴ TIMES
CLAY PARTICLES
FLOCCULATED COLLOIDAL PARTICLES OF LOW DEGREE OF CONSOLIDATION
FLOCCULATED COLLOIDAL PARTICLES OF HIGH DEGREE OF CONSOLIDATION DUE TO LOCAL CONCENTRATION OF PRESSURE
Silt grain
Silt grain
Silt grain
Silt grain

Fig. 3—Theoretical structure of clay, explaining increase in compressibility due to remolding by rupture of high-strength paths of stress.

is on the piles but is due to the consolidation of the remolded soil under its own weight.

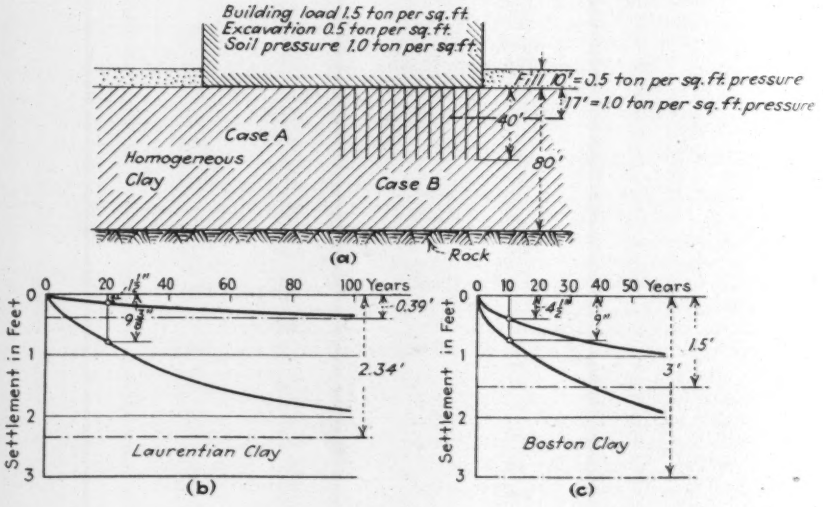
As a specific illustration, let us consider a clay as represented by Fig. 2, which was consolidated under a pressure of about 114 lb. per sq.in. and which by erosion has lost most of the overburden and is exposed to a pressure of about 14 lb. per sq.in. If the load is increased by the weight of a building to 28 lb. per sq.in., the soil will gradually compress from a void ratio of $e_1 = 1.050$ to $e_2 = 1.040$. If the clay were completely remolded, it would have to undergo a compression under its own weight from $e_1 = 1.050$ to $e_3 = 0.865$, and due to the additional load to $e_3 = 0.805$. For the remolded state the compression would be 24 times larger than for the undisturbed state, or, with proper correction for load distribution along the piles and for part disturbance, about eleven times.

To visualize further what these figures mean, the foundation shown by Fig. 4 is presented. Case *A* represents a mat foundation without piles, and case *B* a pile foundation with a 40-ft. embedment in clay. The relative settlements are shown by the time-settlement diagrams.

Reducing settlement

Is there a way to reduce the settlements without reducing the building load? Such a method has already been employed in Fig. 4, without paying special attention to it. The building load is 1.5 tons per sq.ft., but on account of a 10-ft. excavation the pressure on

Fig. 4—Time-settlement curves for raft and pile foundation in clay having the compression curves of Figs. 1 and 2.



the clay is reduced to 1 ton per sq.ft. If the building were put right on the surface of the ground, the pressure on the clay, and therefore also the settlements, would be 50 per cent greater. By excavating more soil the settlements can be reduced to any desired amount. An excavation of 27 ft. would lighten the load by exactly the amount of the building load. That means, after construction of the building, the pressure on the clay would be the same as that under which it is consolidated. Consequently there would be no further consolidation or settlement (provided no piles were driven into the clay).

The fundamental principle, then, for the design of building foundations on

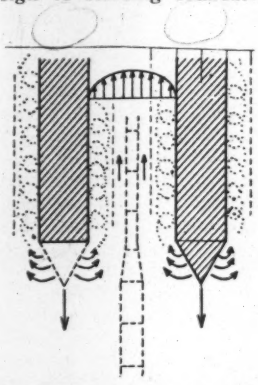


Fig. 5—Remolding action in clay displaced by driving piles.

clay, where firm rock, sand or gravel are not within reach, is to lighten the load by excavation, so that the settlement due to consolidation of the clay will not exceed an arbitrarily fixed limit.

Application of the principle

Based on the research made and corroborative evidence furnished by actual foundations, the opinion is reached that a foundation in clay must be designed so that for any possible stress distribution no harmful differential movements can occur in the building. This does not mean that the building must not settle, but it means that the foundation should be rigid enough to prevent any appreciable bending. A foundation slab alone cannot accomplish this in an economical way. In order to make a raft foundation rigid enough and at the same time not excessively heavy, it must be designed as a truss, combining as its members the whole basement. The system that seems best suited for this purpose is the Vierendeel truss, consisting of rigid frames, as it was for the first time applied in the telephone building in Albany, N. Y. (*ENR*, Nov. 27, 1930).

In conclusion, the whole problem of building foundations on clay boils down to these two simple principles: (1) Do not disturb the natural structure of the clay; if you do, no human being is able to restore its original strength. (2) Decide on a certain rate of settlement which you do not wish to exceed, and determine that pressure which will cause this rate of settlement; the difference between the building load and the above pressure is the weight of soil that must be removed before erecting the building.

A definite bearing value of clay does not exist. As long as engineers are guided by building codes containing definite bearing values for clay, they are consciously or unconsciously guessing, without any assurance in their own minds that they are guessing correctly.

* * *

III—Foundation Soil Overloading Shown by Penetration Test

By Henson K. Stephenson and E. B. Feingold
Assistants in Soil Mechanics Research, University of Michigan, Ann Arbor, Mich.

THE failure of the foundation under the main tower footing of St. Anthony's Church, in Toledo, Ohio, caused serious cracks in the superstructure and consequently impaired the structural safety of the building. The church, feeling that the cause of the failure lay in the construction of a trunk-line sewer in front of the building, entered a suit against the city of Toledo and the general contractors. The contractors felt that the settlement of the tower was due to an overloaded condition of the foundation, and that the construction of the tunnel contributed a negligible source of trouble. To decide the relative importance of the two factors required a complete scientific study of the soil conditions. For this purpose the contractors retained Prof. W. F. Housel of Ann Arbor, Mich. The analyses described herein were made under his direction. They were made first to determine the bearing capacity of the soil and the stresses produced by the church structure in the underground; and second, to investigate the possible effects on the underground due to the construction of the tunnel.

There are two steps in determining the allowable bearing values to be used in the design of a foundation. First, it is necessary to know what stresses will be set up in the earth under the action of surface loading. This problem has been solved both by theoretical and experimental means by Boussinesq in 1893 and by numerous experimentalists, chief of whom are Kögler and Scheidig, of Germany. The second step is to determine just how and to what extent the material under consideration develops internal resistance to external loading. On the basis of recent investigations it is possible to accomplish this.

Bearing capacity

All materials achieve equilibrium under the action of external forces by developed pressure (resistance to volume change) and shear (resistance to the relative displacement of the individual particles that comprise the material). Most of the materials used in engineering (steel, wood, glass, etc.) possess one of these sources of strength to such a marked degree that the other may be neglected. In soils such as the plastic clay (Fig. 2), encountered in this particular problem, it is necessary to evaluate both these factors, as clay acts like a solid in the lower ranges of loading and like a liquid in the upper ranges.

Maintaining equilibrium

For such loads as permit solid action, equilibrium can be maintained under the two stress reactions; in the higher ranges of load the shearing resistance is definitely overcome, and a state of flow (or progressive settlement) obtains. Bearing capacity can then be defined as that load which can be sustained on a bearing area of a given size

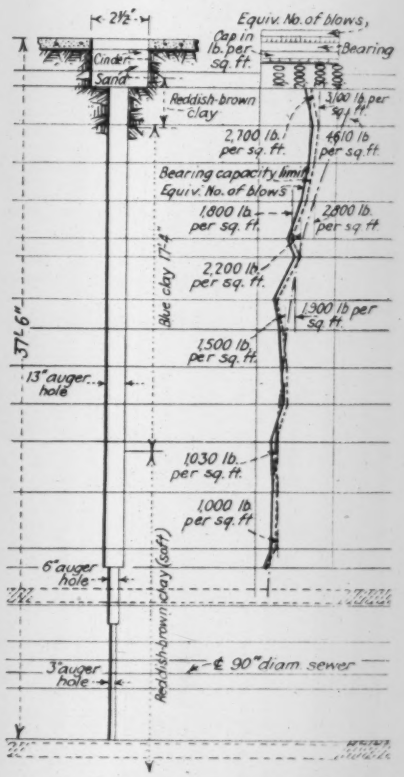


Fig. 1—Graph of penetration test with penetration, bearing capacity and actual soil-pressure curve determinations.

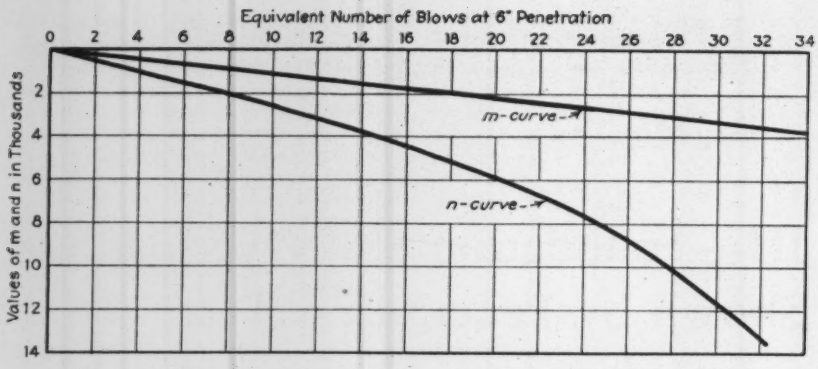

Equivalent Number of Blows at 6" Penetration
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34
Values of m and n in Thousands
2 4 6 8 10 12 14
m-curve
n-curve

Fig. 2—Relation between penetration and values of shear and pressure resistance at bearing-capacity limit.

and shape without producing a state of progressive settlement.

Then for any constant settlement:

$$W = p\,A = m\,P + n\,A,$$

$$\text{or } p = m\frac{P}{A} + n$$

where p is the bearing capacity for a given settlement, P is the perimeter of the bearing area, A is the area of the bearing plate or footing, and m and n are the ability of the soil to develop resistance to perimeter shear and developed pressure respectively.

Values P, A and p can be measured in a series of loading tests upon plates of different size, and values of m and n can be determined by solving the simultaneous equations resulting from the data thus obtained.

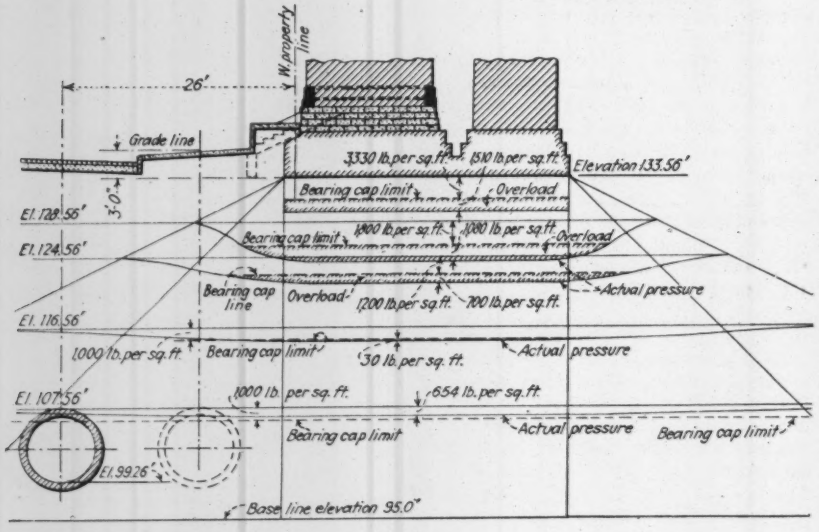
By measuring, then, the loads that will be sustained without progressive settlement on plates of different sizes it is possible to evaluate the two stress reactions at the bearing-capacity limit. Many tests made by Professor Housel and others verify exactly this statement and conception.

Penetration test

At the same time that these tests were made it was thought that a simpler and less cumbersome method of evaluating these reaction constants would be valuable. Penetration tests, using a standard 1¼-in. core-sampling pipe and a driving apparatus consisting of a constant weight falling through a constant distance, were conducted to establish a relation between the two types of tests. At the same elevations at which loading tests were made penetration tests were also carried out, it being necessary only to measure the number of blows required to produce some standard penetration. A study of the data thus obtained revealed a positive relationship between the two methods of testing. This relationship is empirically expressed in Fig. 1. With this information, then, it is only necessary to conduct penetration tests to determine the bearing capacity of any soil for a given size and shape of foundation footing.

Using this method, tests were made in two pits located as close to the tower footing as possible. From the results it was possible to compute the bearing capacity of the soil at different levels below the surface (Fig. 1). It was then necessary to determine the actual stress conditions at these levels as caused by the load of the building.

Fig. 3—Graph of distribution and intensity of soil loading under church tower as determined by penetration tests.



The results of many investigations show that the pressure spreads itself through the underground, creating a bulb of pressure. The limits of this bulb make an angle of about 63 deg. with the vertical. The pressure is distributed on any horizontal plane in the manner of a probability curve. This curve of distribution can be closely approximated by a straight-line variation.

This approximate method was used in the present problem to determine the stresses on planes at different depths. These stresses were then compared with the estimated bearing capacities for the respective elevations. The comparison (Figs. 1 and 3) showed a condition of severe overloading to a depth of about 20 ft. below the bottom of the footings.

Effect of tunnel

Up to this point the analysis merely showed that the foundation was overloaded and might be expected to settle without the introduction of external influences. It was then desired to determine what effects the construction of the tunnel might have had. The amount of squeezing encountered in an excavation is a function of the size of the bore. Such a condition of stability is the result of shearing resistance.

In this case the average shearing strength of the soil was found to be about 340 lb. per sq.ft. Knowing the overburden and the stresses caused by the church, it was possible to compute the allowable (unsupported) length for a tunnel 9 ft. in diameter. This was found to be 15.6 ft. The contractor successfully proved that the tunnel was never unbraced for a distance even approaching this allowable drive, and the suit was denied by the court.

Snow Surveys to Be Made in Australia

Investigations for a proposed 85,000-kw. hydro-electric development in Victoria, Australia, are to include snow surveys following the system used for several years in the Sierra Nevada range of Nevada and California. A part of the drainage area is situated at El. 5,000 to 6,000, where there is a snow cover of 5 to 25 ft. for about six months of the year. Equipment similar to that developed in Nevada has been acquired, and similar methods will be followed in making the forecasts or predictions of runoff. Automatic-gage records of streamflow have been maintained during the past six years, and it is proposed to establish a permanent meteorological station in the basin for the measurement of evaporation from snow and ice surfaces and the seasonal variations of groundwater flow, and to carry on other special work in addition to collecting the ordinary meteorological records. These surveys and studies are to be made by the engineers of the State Electricity Commission of Victoria.

Automatic Sector Gates for Coolidge Dam Spillway

First installation in the United States of automatic control gates of a design originated in Europe—Reservoir capacity increased 21 per cent

By Paul Baumann

Chief Designer, Quinton, Code & Hill—Leeds & Barnard, Los Angeles, Calif.

PROVISION for securing the reservoir capacity represented by the depth of water that can be held above the fixed crest of the spillway of a dam during periods of low flow presents a particularly fertile field for the ingenuity of the designer of hydraulic structures. From simple hand-placed flashboards, the development of this important accessory to dam building has been improved and elaborated as to features of design and control. The present article describes the initial installation in the United States of a concrete-sector type of spillway gate with notable control characteristics. Developed by Barrages Automatiques Société Anonyme, Zürich, Switzerland, specialists in this field of hydraulic design, this type of gate has had several foreign installations since the first one was built about 1916. The author acted as the local representative of the designers for the Coolidge Dam installation. The description will be of special interest to those in the field of hydraulics and dam design. —EDITOR.

AUTOMATIC-SECTOR spillway gates have been added to Coolidge Dam and will add 205,000 acre-ft. (21 per cent) to the storage capacity provided by this structure, built in 1928 by the U. S. Indian Irrigation Service on the Gila River in Arizona. The six gates are of reinforced concrete and operate from a chamber within the dam crest, using the effect of buoyancy to control a 12-ft. head of water above the crest by automatic regulation. It is the first use of this type of gate design in the United States, although the device has been installed on several foreign dams.

The Coolidge Dam is of unique multiple-dome design (*ENR*, Sept. 13, 1928, p. 396, and Sept. 20, 1928, p. 438). At the time of construction, spillways, without regulating gates, were provided on both sides of the canyon with fixed sills at El. 2,511. Each of these spillway openings is 166 ft. wide, with two intermediate piers supporting arches for the roadway over the structure. Recent installation of the six gate units in these openings will provide for controlled raising of the reservoir surface 12 ft. to El. 2,523. Spillway capacity was based on a discharge of 50 per cent in excess of the highest flood of record, which is 130,000 sec.-ft. In discharging this estimated maximum flood of 195,000 sec.-ft. the water surface in the reservoir would rise to about El. 2,531.4, or 20 ft. above the fixed sill.

Increase in reservoir capacity provided by the gates, based on a value of $2 per acre-foot of storage, shows that the $125,000 investment is sound, provided the 205,000 acre-ft. of storage produced by the spillway gates becomes available within twenty years after the installation. Hydrological studies based on runoff records of the U.S.G.S. indicate an average frequency of full reservoir (El. 2,523) once in nine years.

Design of the gates

In each of the six spillway openings a floating-type automatic-sector gate of reinforced concrete is installed, and each unit of three gates is controlled by a mechanism installed in one of the intermediate piers. Gates and operating mechanisms were designed by Barrages Automatiques Société Anonyme, Zürich, Switzerland, a firm that has specialized in automatic gates of various types for twenty years.

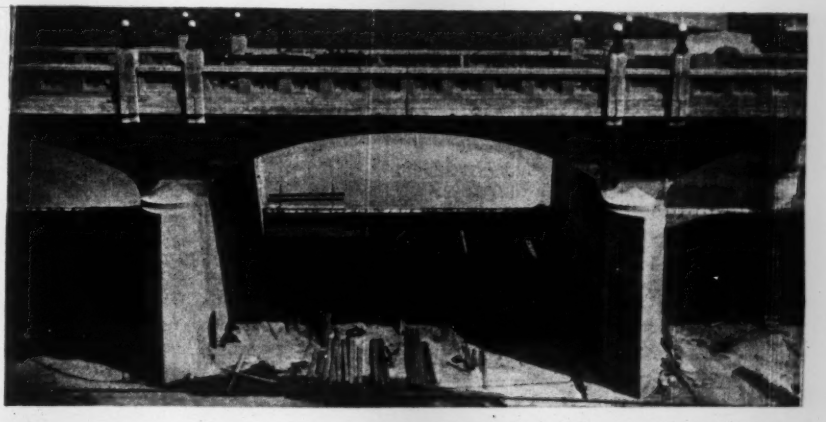


Fig. 1—One of the central 50-ft. spillway openings, with the turning hinge in place and forms for the sector gate under construction.

Functional Characteristics — Before describing the technical details of the automatic sectors it is helpful to consider their operation in handling floodflows. In general, critical floods of the Gila River occur during the winter (December to March) when the reservoir is relatively low after a season's draft for power and irrigation. With the occurrence of such a flood the reservoir will fill, and theoretically, when the water surface reaches El. 2,510.33, or about 1 ft. below the fixed sill, the gates will float and begin to rise from their recessed position. As the water surface continues to rise, the gates will also rise, but at a greater rate, with the result that when the water surface reaches El. 2,518.54 the gates will be in their highest position, with the crest at El. 2,523. Thus, the freeboard of the gates increases from 1 ft. at the beginning of flotation to nearly 4.50 ft. at the highest position. The intermediate relations are shown in Fig. 7, which also shows the result of the actual test as compared to the calculated curve.

As the water surface continues to rise above and to overtop the crest of the gates, the regulating mechanisms begin to function and the gates will slowly lower and provide sufficient spillway opening to make the discharge equal to the flow into the reservoir. Consequently, each increment to inflow will cause a further lowering of the gates, while the water surface remains substantially constant.

At the instance the gates reach their lowest position again the water surface will be at El. 2,523.67 and the flow into the reservoir will be 45,000 sec.-ft., with a corresponding head of 12.67 on the spillway based on a discharge coefficient of 3.33. This factor was established by tests on models in the hydraulic laboratory at Karlsruhe, Germany. Each succeeding increment in floodflow will cause a rise in the water surface and an increase in the discharge head.

Upon passing the final flood peak, the process will reverse itself, the water surface dropping to El. 2,523.67 before the gates commence to rise. The action will then continue until the gates reach their highest position as the inflow ceases or equals the draft and losses. The sectors may be kept in any position by means of manual control.

Form of sectors

Each sector is a closed reinforced-concrete compartment 45 ft. 4 in. long, with a central angle between faces of top and bottom slab of 41 deg. and a radius of the upstream face of 17 ft. A section through one of the sectors is shown in Fig. 3.

Seven cells are formed in each gate by the two end walls and six partitions spaced 6 ft. and 6 ft. 6 in. apart. The

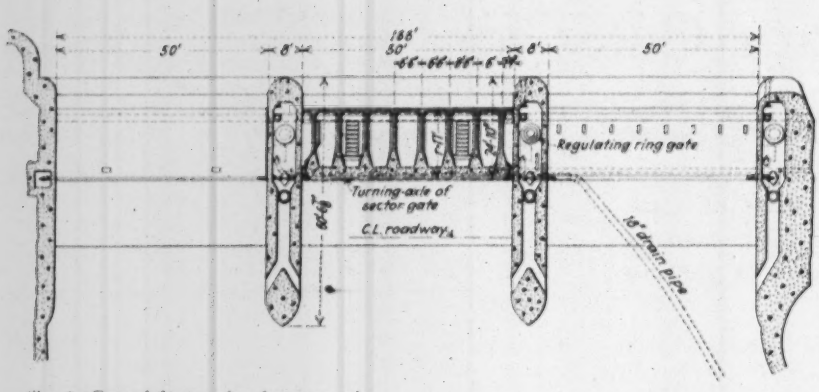


Fig. 2—General layout plan for west unit of three gates. Central gate, in section, shows location of diaphragm walls. Control equipment is in right-hand pier.

difference between the length of the sector (45.33 ft.) and the 50-ft. opening is made up by a cantilever extension of top and face slab of 2 ft. 2½ in. and a seal of 1½ in. at each end. This construction is provided to make the end seals accessible. The diaphragms and end walls support the three main slabs, which are designed as continuous. As a result, the water pressure on the face slab is transmitted to the hinge and the sill, principally through these walls whose tapering sections help to distribute the load.

The gates are hinged downstream and the thrust is resisted by a massive steel and reinforced-concrete sill. To prevent the gate from moving upstream when in a low position, retention hinges somewhat similar to door hinges are provided near the ends. Between these hinges and the piers or abutments the shaft is circular and hollow.

The main or thrust hinge is designed to transmit the thrust to the sill and to seal against leakage from the gate chamber without producing appreciable rotary friction, which is an important feature with moving gates. The true setting of this sill, both as to alignment and elevation, is the key to successful operation of the gates.

To prevent leakage out of, as well as into, the gate chamber, spring-steel end seals are fastened to the top slab and the face slab, and, finally, a similar seal is provided on the steel casting on top of the breast wall extending across the face. This latter seal and the end seals along the face are only of service in preventing water from entering the gate chamber. However, the end seal along the top slab is designed for double action: when the gate is up and there is no overflow its purpose is to keep the water from getting out of the gate chamber; whereas, for the gate lowered and water discharging over the top its function is reversed, because in the latter case the outside pressure may exceed the inside pressure. The seals are designed so as to become tighter with an increase in water pressure.

To assure efficient sealing, especially against loss of water at the ends, the faces of the piers and abutments were required to vary not more than plus or minus ⅛ in. from a plane surface. This allowance was not exceeded on the Coolidge spillway, and by means of templets and a specially designed level compass the cement plaster surface on the cylindric face was kept within the same tolerance.

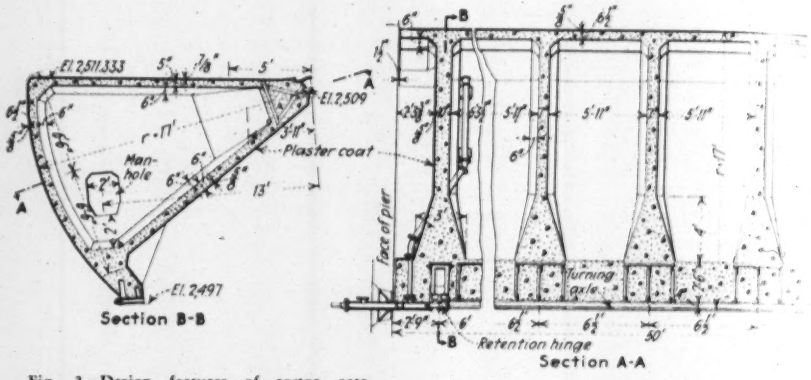
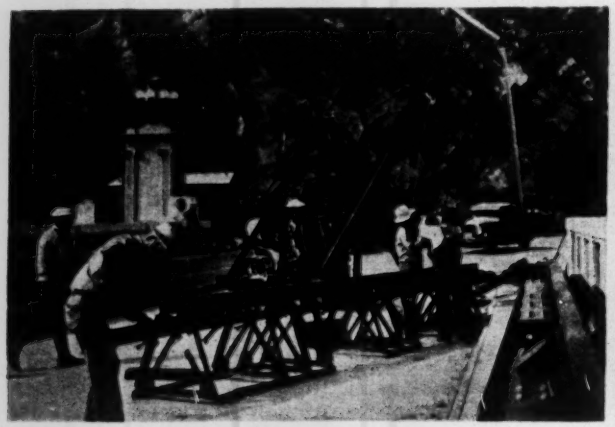


Fig. 3—Design features of sector gate shown by two sections. Water faces are covered with coat of cement plaster.

To drain the interior of the sector in case of water percolating through the concrete, two separate systems, one for a low and one for a high position of the sector, are provided. The first consists of a pipe extending to the bottom, which runs through the hollow shaft into the hollow pier where suction by ejector or pump may be applied. The second is a short pipe and vent extending from the hollow shaft to the inside of the sector at the hinge side. In raised position the bottom of the sector slopes downstream, with the result that this drain will function automatically.

The water in which the sector floats is confined in the gate chamber and is admitted through 12-in. pipes, one in each intermediate pier. The openings of these pipes are controlled through valves from the top of the piers.



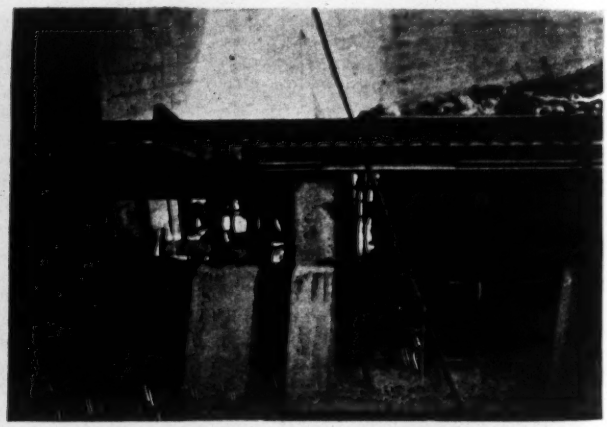


Fig. 4—Assembling of turning axle (left), which is continuous between retention hinges, and (right) end of axle in position, showing retention hinge and provision for penetration of pier wall.

The breast wall that forms the upstream confinement of the gate chamber controls the upward motion of the sector. A steel shoe along the bottom of the sector seals against a continuous stop, which is a part of the steel casting bolted to the top of the breast wall. The maximum theoretical uplift for water surface at El. 2,523 is 2,025 lb. per linear foot. Bolts and steel casting were designed for double this load to meet possible non-uniform bearing.

In its lowest position the sector rests on steel supports, one under each end and partition wall. The gate chambers of each unit, which are intercommunicating, may be drained through an 18-in. pipe connecting the sumps in the intermediate piers and discharging into the spillway channel below the gates.

Gate operation

The operating mechanism controlling each unit, as previously mentioned, is installed in one of the intermediate piers. All of its moving parts are sturdy, simple and positive in action; it is, so far as humanly possible, foolproof. The energies necessary for its operation are from two sources: (1) the float and (2) the sector gate. The control mechanism is shown in Fig. 5, and its principal features will be briefly described.

The raising of the gates during initial rise in the water level above El. 2,511 is a process of simple flotation and does not require the functioning of the control mechanism. However, when the water surface reaches El. 2,523 and the gates are in their raised position, the float begins to lift and by means of a leverage system (Fig. 5) causes the cylinder gate to rise, allowing a discharge of water from the gate chamber.

As a result, the pressure under the gate decreases and the gate starts to lower, turning a sector-shaped lever that is fastened to the gate shaft. This latter action has a counteracting effect, to prevent too rapid dropping of the gate, acting through the wire rope and shaft-lever system from which the cylinder gate is suspended. This second force produced by the lowering of the sector tends to balance the opposite action of the rising float, permitting the cylinder gate to drop back to such an opening that the discharge equals the inflow. The pressure under the gate will then be constant, and the gate will come to equilibrium. This process repeats itself for each additional rise in water surface, with the discharge through the cylinder gate gradually increasing until it reaches maximum discharge, when its opening is a little over one-quarter of its diameter. At this instant the water surface in the reservoir has risen 8 in. and the gate is all the way down. During subsequent fall in the reservoir level the operation of the control mechanism is reversed.

The float chamber is connected with the water of the reservoir through intakes located in the corner between the piers and the adjacent domes, where they are not affected by the velocity of the falling water. A 12-in. pipe conducts the water to the operating pier from whence it rises through a 6-in. pipe to the float chamber. The equilibrium condition for an arbitrary gate position is shown graphically in Fig. 6. It indicates the correct condition with the resultant of the forces passing through the hinge and producing a zero moment about the hinge.

The supply line to the float chamber is independent of the intake line to the gate chamber. The latter is installed in each intermediate pier—that is, two in each unit. It supplies the water necessary to maintain equilibrium when the ring gate is raised, or, in other words, it replenishes the water discharged through the ring gate. The pressure under the gate then is equal to the static pressure minus the velocity head minus the friction head; and as both velocity and friction heads increase with an increase in discharge, the pressure under the gate decreases accordingly and the gate drops. Thus, the regulation is done by hydraulic and *not* by hydrostatic pressure, as commonly assumed.

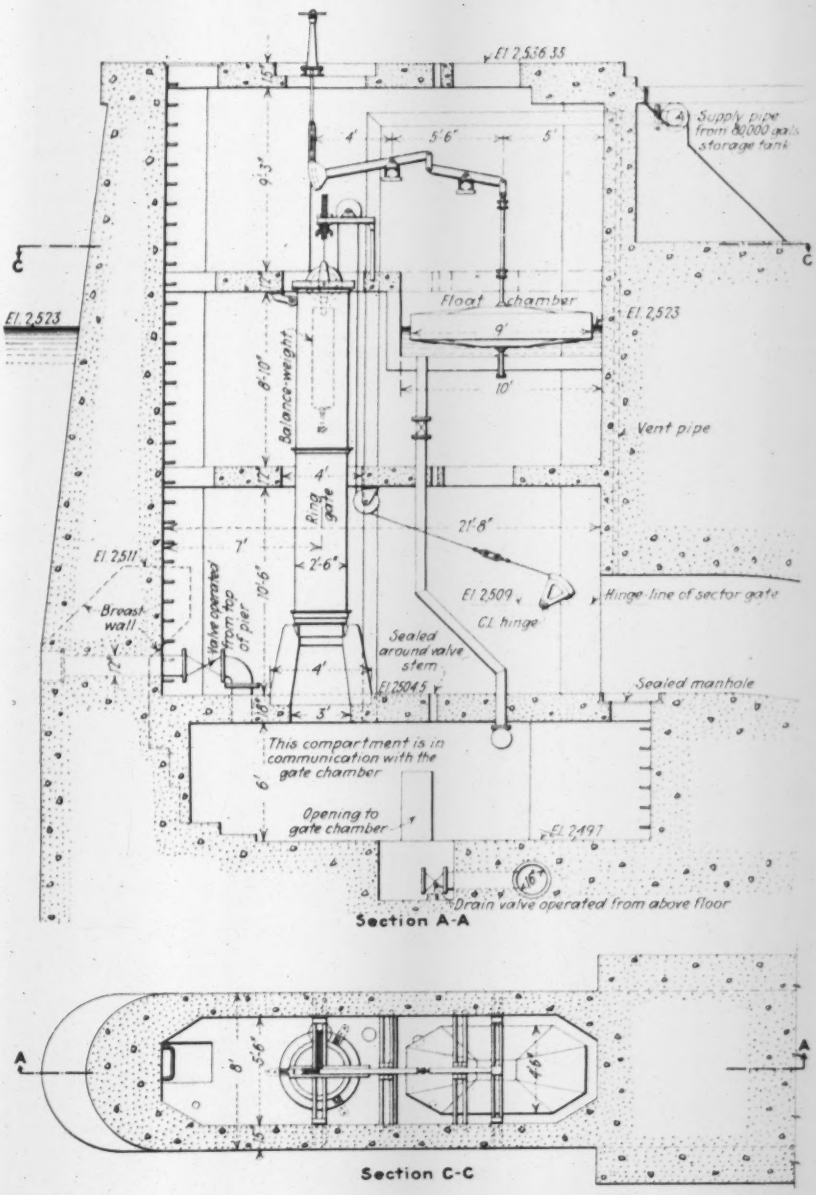


Fig. 5—Automatic-control mechanism for the gates is located in the piers. Action of float in raising ring gate to lower spillway gate is damped by counter-movement of sector-shaped lever and cable activated by lowering gate, so that balance is secured. Otherwise, rising reservoir level and float action would produce rapid drop of sector gate and sudden flood peak below dam.

Construction

The construction operations involved some unusual problems, including the complicated formwork, the pouring of

the curved face wall and the rigid requirements for producing plane surfaces. These difficulties were completely overcome as the work progressed. Wooden forms were used, built up to fabricate the gates in their lowest position. Manholes in the diaphragm walls and top slab permitted removal of the inside formwork.

Concrete was 1:1¼:2½ mix with a 6-in. slump. Aggregates left over from

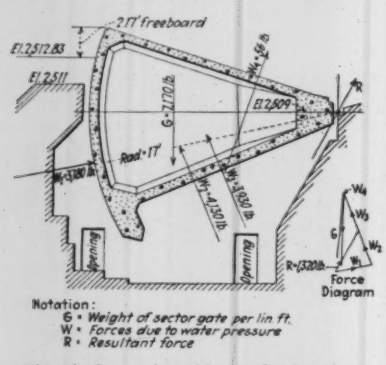


Fig. 6—Sector in arbitrary floating position maintaining freeboard, showing resultant of forces passing through hinge with zero moment.

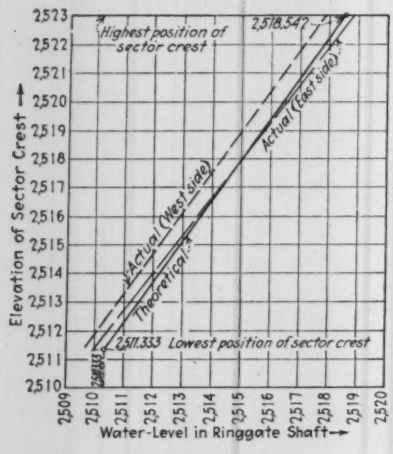


Fig. 7—Results of actual tests of gate action compared to theoretical rise. Diagram indicates the progressive stage of freeboard maintained by rising gate.

the dam were used and proved satisfactory, the 28-day tests showing an average strength of 4,153 lb.

Tests on the gates

Tests were made on the gates in January to check the operation and control. During the first test the end seals showed excessive leakage, from 60 to 70 g.p.m. per seal, but subsequent tightening resulted in reducing the leakage during the second test to 20 g.p.m. per seal or less, which is an amount to be anticipated.

As the water in the reservoir was about 100 ft. below the spillway at the time the tests were made, a wooden bulkhead was built on top of the breast wall to retain the water necessary for floating the spillway gates. Fig. 7 shows the results of the tests and indicates that the west gates were somewhat lighter than estimated by the designers, whereas the east gates very nearly checked the computed characteristics. This is probably caused by the latter being less dried out at the time of the tests. The difference in slope is due to friction in the end seals and the resulting lag.

The writer wishes to acknowledge the assistance of A. L. Wathen, supervising engineer, fourth district, U. S. Indian Irrigation Service, and his assistant, E. C. Fortier, who was in direct charge of the work, for valuable assistance in preparing this article.

The construction was carried out by Bent Bros., Los Angeles, for a contract figure of $87,000, under the direction of Frank C. Livingston, superintendent.

The automatic sector gates described in the foregoing article constitute the fifth installation of this particular type of control gate designed by Barrages Automatiques Société Anonyme, Zürich, Switzerland. The first installation of this type of gate was made in Switzerland about 1916. In 1924 an installation of this type was made near Cagliari, Sardinia, where the gates provided an effective head of 23 ft. on two 39½-ft. openings. The following year two 89-ft. gates were constructed on the dam near Camarasa, Spain, operating with an effective head of 22 ft. 9 in. In 1926, on a dam on the Parahyba River in Brazil, the largest installation of this type of gate was made when three sectors, 148 ft. long, were built to control an effective head of 24 ft. 4 in. The installation on the Coolidge Dam is the latest installation of this type of sector-spillway gate.

Metal Inclosures for Houses Being Tried in Cleveland

TWO EXPERIMENTS with metal sheathing for houses, one in the form of bent plates, eliminating the necessity for a skeleton frame, and the other in the form of steel-faced panels of fiber insulation boards, hung on a structural-steel frame, are being carried out near Cleveland. The exteriors of both houses are to be faced with porcelain-enameled steel shingles. Both houses are largely shop-fabricated.

In the house being built in Solon, by the Insulated Steel Floor & Wall Co., which is affiliated with the American Rolling Mill Co., steel sheets formed into vertical Z-bends, with prevailing dimensions of 11 in. for the flanges and 6 in. for the web, will constitute the framework. Mills G. Clark, of Cleveland, developed the design. These Z's, when assembled in the wall, produce a corrugated-box section, the troughs of which will be filled with mineral wool. The wall will be faced inside and outside with insulation board, while on top of the outside insulation porcelain-enameled steel shingles will be applied. The floor system will be built up of Z-shaped plates overlapped and welded together to form a series of box sections, which also may be filled with sound-deadening insulation. It is contemplated that 16-gage metal will be used in the floors, 20- or 18-gage metal in the exterior walls and 20-gage metal in the interior walls. This house will be entirely welded, part of the work being done in the shop and part in the field. The foundations of this house have recently been completed.

The other steel-sheet house is being built in South Euclid by the Ferro-Enamel Corp. It is practically finished and comprises a bolted steel frame, welded bar-joist floors and an exterior inclosure of Truscon "ferro-clad" panels made of 24-gage copper-bearing steel sheets gummed to the sides of ¾-in. wood fiber board. On this base the porcelain-enamel steel plates vulcanized to asphalt shingles have been nailed. Mineral wool is to be used for insulation in the air spaces of walls and floors.

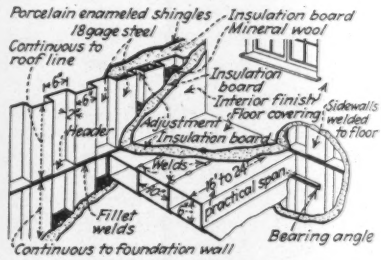


Fig. 1—Typical construction of frameless steel house in Solon, a suburb of Cleveland.

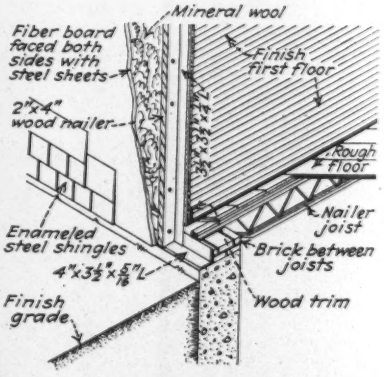


Fig. 2—Enameled steel shingles on steel-clad insulation boards form the exterior walls of a house under construction in South Euclid, a suburb of Cleveland.

Building Change Orders and General Costs

Methods adopted on department store unit of Cleveland terminal group to handle change orders without delaying work and to distribute general costs rationally to various subcontractors

By R. P. Wallis
Expediting Engineer, Lundoff-Bicknell Co., Cleveland, Ohio

THE TASK of the general contractor for building construction involves a multitude of details in coordinating the work of the various subcontracting trades and in adjusting common charges between them. On the operation described in this article the task was complicated by making many separate contracts, with the general contractor as the coordinating agent sharing authority with the architect's superintendent. The article describes procedure in the two important tasks of handling change orders and of distributing general expenses. —EDITOR.

THE CONSTRUCTION of the Higbee department store, one of the large buildings of the new terminal group in Cleveland, Ohio, developed a number of construction management practices that were successful in simplifying and expediting the work. Two of these practices relating to change orders and to distribution of costs for shared services merit extended mention because of their importance. In addition, subdivision of the work and an interlacing of double ownership, tenant and double architectural direction produced unusual contract requirements.

Contract conditions

The Higbee store occupies an air-rights area of 70,000 sq.ft. It has twelve stories above ground, a basement at the 72.5-ft. level and an equipment room occupying that part of the 52.5-ft. level not occupied by tracks. The framework is steel, the walls are limestone and terra cotta, the floors above street level are hollow tile, and at street level and below are concrete. The ownership below street level rests in the Cleveland Union Terminals Co. and above street level in the Cleveland Terminal Building Co. The Higbee Co. is tenant. The building was designed by the architects for the terminal group, and all store layouts and fixtures were designed by the architects for the tenant.

The contract for the building steelwork (12,000 tons) was a part of the blanket contract for the terminal group. In addition, there were separate contracts for elevators and inclosures, escalators, dumb waiters, spiral conveyors, paper chutes, track and platform work, special lighting fixtures, store fixtures and decoration. The remainder of the work was let as a general contract, and the general contractor assumed coordination of the majority of the extra contracts. This interrelation of responsibility and authority produced the special need for the control methods described.

Change orders

The subcontractors were required by the specifications to submit a schedule of unit prices in conformity with a list of items enumerated in change orders. The unit prices included only labor, material, insurance, plant, rental of equipment and the use of tools and were to apply to deductions as well as to additions. No overhead, profit or general expenses aside from insurance were to be included.

Upon completion of the contract the extras and credits of each contractor were totaled. Should this summation indicate a net extra, the contractor was allowed a previously agreed-upon percentage upon the overrun only. Should credits exceed extras, no percentage was allowed the contractor for his extra work. Where unit prices could not be made to apply, changes were carried out under a lump-sum estimate or on a time and material basis. All time slips and material receipts were to be signed daily by the architect as substantiation of the contractor's subsequent claims for additional compensation.

Changes in construction were initiated with the architect, both by his main office and by his representative in the field. Their instructions came to the contractor in the form of (1) revised drawings, (2) letters of instruction and (3) verbal directions confirmed by either memos from the architect's superintendent or by reference in the minutes of weekly job meetings.

Upon receipt of instructions calling for change, copies of blue prints or letters were distributed among the several contractors interested. Where the work was to proceed at once, the subcontractors were ordered to place the work in immediate production by means of a stamp applied to each print or copy of the letter. In the absence of a proceed order the subcontractor was merely requested to submit his estimate for the consideration of the architects and owners.

These estimates were prepared on special printed forms provided by the contractor. The individual estimate sheets of the various contractors interested in a particular change were assembled by the contractor's change-order clerk on a summary sheet for the consideration of the owners. These pilot orders, as they were designated to distinguish them from the subequent formal submission, bore a change-order number assigned by the contractor. These estimate sheets might list only unit-price items, or lump-sum estimates or a combination, depending upon the class of work affected. Time and material items were for the most part consolidated into composite orders by contractors rather than by operations.

The owner, upon receipt of the pilot orders, would check them for price against the market and against the drawings for quantity. They were then returned to the contractor bearing whatever corrections the owner felt himself entitled to make. Should the subcontractor accept such price adjustment, the estimate sheet was rewritten in final form and submitted by the contractor for the formal signature of the architect and owner. Otherwise further negotiations ensued.

Upon receipt of the formally approved order a purchase order was issued to the various subcontractors in the amount of their respective estimates, authorizing the subcontractor to place the order on his books as a charge or credit against the job. In the meantime the work involved may have been entirely completed, thus permitting the uninterrupted progress of the job.

Cost distribution

Another interesting feature of this contract was the method devised by the architects for the equitable distribution of the costs of certain items among the various contractors involved. Such incidentals as watchmen, electric light and power, plaster patching, cleaning and temporary elevator service are all in some measure the interest and concern of each of the sub-trades, and each should bear a share of the expense commensurate with the benefit derived.

In this particular contract the responsibility of arriving at a fair distribution of costs was placed with the architect's superintendent. The work in each case was to be performed by the trade under whose jurisdiction it would naturally fall, under the direction of the architect's field office.

Costs were to consist of labor, material, insurance and other direct expense. An allowance of 6 per cent for overhead was included in each case, but no profit was included either to the individual subcontractor or to the general contractor. Funds to cover these costs were advanced by the general contractor, who was from time to time reimbursed by the sub-trades. The steel fabricator and erector and the hardware contractor were exempt from these provisions.

To make these assessments as fair as possible, direct charges were considered separately from general charges. The former comprised those items directly traceable to individual trades and were

assessed direct by the architect's representative. The latter consisted of such items as did not admit of individual responsibility.

Individual accounts were kept by the architect and contractor. The final results were arrived at by the architect after comparing these accounts and after a careful review of all the circumstances.

The watchmen were hired by the contractor. The costs, comprising payroll, insurance and percentage, were split up among the contractors on the job on the basis of the percentage of individual subcontracts to the total contract. Those contractors furnishing material f.o.b. Cleveland were included in this prorating, as they had not been exempted by the specifications. Furthermore, it was the opinion of the architects that their responsibity did not cease until their material was safely incorporated in the structure. These costs were billed to the subcontractor in four installments and covered watchman service for the entire period of the job.

Temporary light and power were in reality two separate items. Power was handled as a direct charge as well as that portion of light involved in the lighting of the contractor's shanties. The remainder of the costs of temporary lighting were distributed among the various subcontractors on the basis of the probable need on their part of artificial light during the construction period. The lighting of stairways and other passages was grouped separately and was assessed on all of the subcontractors irrespective of their other light requirements. These assessments included labor, materials, insurance, electrician's percentage, illuminating company's service charges and current consumed. Two separate billings were rendered on this account.

Plaster patching executed by the plastering contractor was likewise segregated into direct and general. The former was assessed as a back charge wherever it was possible to ascertain who was responsible for the patching. General patching was divided up on the basis of probable responsibility, including all of the contractors as far as stair halls were concerned. This account, including labor, materials, insurance and plasterers' percentage, was covered in three installments.

In the case of temporary elevator service the costs of placing the elevator in service, labor, material, insurance and the contractor's percentages thereon were covered by the elevator contractor. The operating power and maintenance costs were taken care of by the general contractor. Costs were prorated on the basis of man-working days, consideration being given to those crafts not using the elevators in the normal course of their employ.

Cleaning was a large item. Here again it was possible to segregate the costs into direct and general. Time slips covering both classifications were signed daily by the architect's office. Assessments were based on the man-working days. Direct cleaning was back-charged against the contractor held responsible and billed monthly, while general cleaning was invoiced in two assessments.

In addition to these items covered by the specifications, there was a number of similar operations pro-rated by the general contractor in a manner similar to that above outlined but without being subject to the architect. The construction cost of a road at the track level was pro-rated among the various contractors, using this facility on the basis of a count of incoming trucks. Water for construction purposes was handled in similar manner. The needs of each contractor were analyzed, and his proportionate responsibility was determined, but a minimum charge was assessed all contractors for general water use.

The architects on this project were Graham, Anderson, Probst and White, of Chicago, and the contractor was the Lundoff-Bicknell Co., of Cleveland and Chicago.

Property and Street Map Surveys Give Engineers Employment

By L. G. Merrill
City Engineer, Parkersburg, W. Va.

PREPARATION of property and street maps for the city of Parkersburg, W. Va., and for two of the ten magisterial districts of Wood County (districts are comparable to townships in other states) has enabled the writer to give continuous employment to five or six engineers when they were not otherwise engaged on engineering work. Moreover, the cost, which was partly paid for by the county court of Wood County, has been offset by discovering and getting on the tax rolls property that heretofore had escaped taxation. The added taxes collected for one year from this source were sufficient to pay the cost of the entire work.

The maps, which are based on the plan adopted by Washington, D. C., show every lot, tract or parcel of land with their dimensions, lot numbers and the subdivisions to which they belong, together with all the material improvements. Streets, alleys and roadways are shown. They simplify the public records relating to titles and will make available a fund of information that is of great economic value to the county and city. So far as the work has progressed, it has resulted in securing the taxation of all real estate and the improvements.

Index cards with the number of the lot, a brief description of the subdivision, together with the owner, date of purchase and deed-book reference, were provided for each lot or parcel of land. Tracts larger than lots are assigned letters for identification. On the scale used, 1 in. equals 50, 100 and 300 ft., respectively, for business, residential and farm lands.

Before beginning actual work on these maps, tracings were made of all subdivisions of record, descriptions of all tracts of land were copied, photo copies were made of assessment records, and prints of other properties in the possession of independent engineers were purchased. Control surveys and traverses were made of highways.

Each sheet covers an area of 18x24 in., less space for margin and binding. It was not possible to make block maps, as is sometimes done, on account of the irregularity of the street system. Ordinarily each sheet is made up of several subdivisions and unrelated parcels of land. The original lot number is shown in a small circle. In addition to the original lot number, each lot is assigned a distinctive number to identify it on the index card.

Where the prints are handled a great deal by the public, they are reproduced on a heavy white muslin and bound in substantial loose-leaf binders.

Buildings are colored a distinctive color to represent the different types of construction. Symbols are used to denote the type of structure and the number of stories.

It is the intention to keep maps and property transfers up to date. New additions or subdivisions will be added as they are placed on record. The county court has signified its intention of adopting these maps as its official record. If this is done, property could be transferred merely by giving the lot and page number—or, as it is called, the block number.

Aside from future value of the plats for city planning, zoning and traffic studies, the maps have proved of great value recently in making studies and preparing estimates and contract plans for a 19-mile water-distribution system and a 16-mile sanitary sewer system. The maps were used to good advantage recently when the city and county court revamped their voting precincts. A new territory equal in area to the old city was recently annexed. A set of reproduced tracings was used to compile a house-numbering system of this area. A number was assigned to each lot. The makers of insurance maps have used these maps for extending and enlarging their plat books. Several local utilities, including gas, electric light and power and telephone companies, have used the plats for making records and preparing estimates and plans for future work and extensions.

Driving Bridge River Tunnel, British Columbia

Three-mile bore for hydro-electric development of British Columbia Electric Railway Co. encounters heavy ground—Contract provides pay for overbreak concrete at fixed rate—Grouting data

By C. E. Blee

Principal Assistant Engineer, Construction Dept., British Columbia Electric Railway Co., Ltd.

PHYSICAL difficulties of considerable magnitude and special contract provisions characterize the British Columbia hydro-electric tunnel described here. Shattered rock and talc that ran like concrete when saturated with water and the accompanying terrific pressures constituted the physical problems that finally were solved by realignment. The contract was carefully drawn in general terms rather than specific requirements to facilitate the necessary latitude of execution so essential to the carrying out of this type of construction. A special provision was the item of payment for overbreak concrete at a rate slightly less than its estimated cost in place. The purpose was to reduce the uncertainty of this item as usually reflected in the bid and still provide incentive for keeping overbreak at a minimum. In addition to the description of the tunneling operation, the article outlines the principal features of this important hydro-electric power development near Vancouver.

—EDITOR.

A THREE-MILE tunnel to develop 1,235 ft. of power drop constitutes the first major construction undertaking on the Bridge River project of the British Columbia Power Corp., Ltd., about 130 miles northeast of Vancouver, B.C. The flow of Bridge River, augmented by future storage, will be diverted through an intervening mountain by the tunnel and used in a power house located on the shore of Seton Lake. The ultimate project will regulate the entire flow of the river to about 3,100 sec.-ft. and provide for an installed capacity of more than 500,000 hp. Notable features of the tunneling operations, completed January, 1932, include the handling of extremely heavy ground, specifications for payment of overbreak concrete and the data obtained in the process of grouting behind the lining.

Development plan

The tunnel that will convey the flow from Bridge River, a branch of the Fraser (Fig. 1), to the power house on Seton Lake is 13,228 ft. long, with a uniform grade of 7.3 ft. per thousand feet. It is concrete-lined throughout and for a distance of 11,220 ft. in from the Bridge River portal has a horseshoe section equivalent to a 14 ft. 3-in. diameter of circular conduit. At the end of this section, about 2,000 ft. from the lower portal, a Y is provided, and one leg was extended to the Seton Lake portal for present development in the form of a circular tunnel 8 ft. 10 in. in diameter. The other leg of the Y was driven some 100 ft. and lined for a sufficient distance to form a support for a steel bulkhead, which will permit its completion for future installation. The ultimate development of the project contemplates paralleling of the present bore by a second tunnel of greater capacity.

Below the Y and 478 ft. in from the lower portal the tunnel is enlarged to a 16-ft. diameter for a distance of 185 ft. and connects with a 7-ft.-diameter shaft extending 320 ft. to the surface to provide a surge chamber. The lower end of the enlarged section will connect with a steel pipe placed in the tunnel and backfilled with concrete.

The tunnel is designed to supply four 56,000-hp. units. As originally planned, it was to be unlined and would supply three 28,000-hp. units with a program that called for enlargement and lining after the construction of the second tunnel. However, a change in the development plans for the company's system and a postponement of the bringing in of the Bridge River project changed the program after the surge chamber and about 1,300 ft. of tunnel had been driven. Because of the bad ground that had been encountered during driving at the lower end it was considered inadvisable to attempt to enlarge this section of the tunnel, so the small section already driven was carried ahead into sound rock where the Y was located. Beyond this point the tunnel was completed to the present enlarged section.

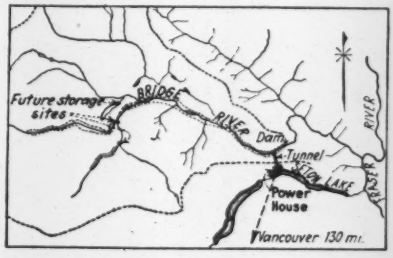


Fig. 1—General features of Bridge River development where tunnel provides a 1,235-ft. power drop to Seton Lake.

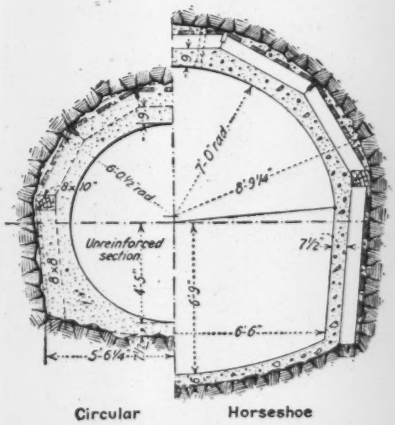


Fig. 2—Half-sections of timbered and lined tunnel. Right, main horseshoe section; left, circular section below the Y.

Tunnel design

For the first 650 ft. downstream from the intake portal the tunnel is of circular section 14 ft. 3 in. in inside diameter. The first 200 ft. has a 24-in. lining, and the remaining distance a 12-in. lining. This circular tunnel form and the thickened lining were used in anticipation of the heavy external pressures that may be imposed upon the empty tunnel by the 150-ft. head of the ultimate Bridge River storage and diversion dam. The rock is comparatively pervious.

No reinforcement was used in the tunnel lining except in the transition section. The 10,557-ft. section between the end of the circular tunnel and the Y has a uniform section of standard hoseshoe shape (Fig. 2). It will be noted that the invert lining is 6 in. thick and the side walls 7½ in., with the arch increasing from 7½ in. at the spring line to 9 in. at the crown. This added thickness at the crown was provided to accommodate the discharge pipe from the concrete gun.

From the Y to the surge chamber a circular section was used (Fig. 2), which was timbered practically all the way, the untimbered section being too short to permit economic change in tunnel section. For all concrete-lined tunnel sections a value of 0.014 for n in Kutter's formula was assumed. The steel pipe of the penstock extends 293 ft. in from the downstream portal to a junction with the surge chamber, thus providing an overburden of rock more than sufficient to balance the internal pressure of the water. The steel lining was justified on account of poor quality of rock and serious damage that might result from excessive leakage.

Surge chambers and shaft are reinforced throughout to carry the full static head plus surge head, with a stress of 30,000 lb. in the steel. Actually the load is considered as divided between rock and reinforcing, so that the stress in the steel will not normally exceed usual working limits. The design of the surge chamber for an assumed head of 300 ft. resulted in the

use of heavy reinforcing consisting of 1¾-in. square bars in the form of circular hoops spaced at 7¾ in. For a distance of 430 ft. beyond the surge chamber the circular tunnel is reinforced with 1¼-in. square bars at 7 in. The reinforcing is arbitrarily reduced beyond this point in accordance with the nature of the rock, and all reinforcing is dropped 1,330 ft. in from the surge chamber, except a short section at the Y.

Driving

Driving was done entirely from the two portals, as there was no location that permitted an adit or shaft. When driving had progressed far enough to indicate that there would be no difficulty in completing excavation within the time schedule provided by contract, the work on the Bridge River heading was stopped so that lining operations could be started from that end. This permitted driving to be carried on entirely from the Seton portal, which had the advantage of drainage and hauling with the grade. About 4,500 ft. was driven from the Bridge River portal, and the remaining 8,730 ft. from the lower end. Some water was encountered at the Bridge River end, particularly when the river was high, but the flow was handled by two small pumps at a sump 1,000 ft. from the portal. The water problem was never serious in this heading.

The formation at each end of the tunnel is volcanic rock, best classified as greenstone, while the interior section (including the major part of the total length) is composed of siliceous sediments. Both formations have been intensely fractured and folded into a syncline with the axis lying near and parallel to the summit of the ridge.

In general, the rock broke well and stood with very little timbering, but a marked exception was some very bad ground encountered near the lower portal. About 1,000 ft. in from the Seton portal the rock became badly shattered and broken into small lenticular pieces, with the seams filled with talc. When water was present in this material it flowed very much like a rich concrete mixture. Increasing amounts of water were encountered, and driving was carried on with great difficulty. Side drifts were carried ahead in an attempt to intercept the water, but these did not prove very effective.

The amount of timbering was increased until 12x12-in. timbers were being placed practically solid. Finally, after traversing 200 ft. of this bad ground a heavy flow of water under high pressure was struck. Its force was so great that rocks and boulders, some of large size, were washed from the heading and carried back into the tunnel. This water seemed to choke its own flow and would then travel back along the tunnel, breaking out in some new point in violent streams. The added weight on the timbers caused by the saturated material, and possibly by some actual water pressure, crushed them progressively back from the heading. In spite of extra sets of 12x12-in. timbers placed inside the regular sets it was impossible to hold the ground, and nearly 250 ft. of tunnel was crushed in and lost.

As a result of this difficulty a detailed investigation of surface geology was made, supplemented by a subsurface survey, using an electrical prospecting method. These subsurface studies (*ENR*, Sept. 4, 1930, p. 364) indicated

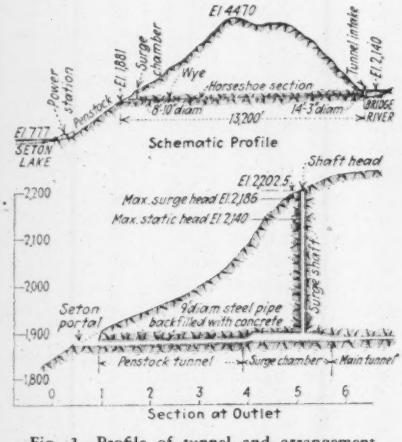


Fig. 3—Profile of tunnel and arrangement of surge chamber.

that the zone of shattered water-bearing rock was considerably diminished about 150 ft. east of the original tunnel line. As a result, it was decided to change the alignment and take advantage of the better rock. An angle of 40 deg. was turned in sound rock about 400 ft. back from the abandoned face. An equal deflection was turned after an offset of 150 ft. had been gained, and after passing the bad ground a slight deflection was made that brought the tunnel line back to original alignment in a distance of about 5,000 ft.

Steel Sets—As a result of the experience in the abandoned heading and in anticipation of bad ground in the offset alignment that could not be held with ordinary timbering, steel sets were designed and provided for 100 ft. of tunnel. Three different types of steel sets were provided to be used in accordance with the nature of the ground encountered:

Type 1—12-in. channels with tie member at bottom below invert (Fig. 4).

Type 2—12-in. channels with footings under posts in place of tie member.

Type 3—10-in. channels with footings.

The first two types were designed for a load of 20,000 lb. per foot of perimeter, or 5,000 lb. per sq.ft. of tunnel surface, with sets spaced 4 ft. on centers. Type 3 was similarly designed for 12,000 lb. per foot of perimeter. All of the steel sets were built so that they could be slipped in between the standard timber sets after the timber was in place, if it was found necessary to reinforce.

Rock conditions conformed closely with the predictions of the electrical investigation, but water pressure was reduced by drainage through the abandoned tunnel. Driving went ahead with less difficulty than had been anticipated. In the worst sections a pioneer drift was carried ahead, and side drifts were kept well in advance to intercept the flow of water. With these precautions and a knowledge of what was to be expected, consistent progress was made. It was found that the ground could be held with timber, and the steel sets were not used.

Normally, all driving was done with the full heading method. Drilling was started from the top of the muck pile and was carried on concurrently with mucking. Air-driven mucking machines were used with steel side-dump cars of 1½-cu-yd. capacity on 24-in. track. Hauling was done by a storage-battery locomotive at one end and by a trolley-type locomotive at the other.

Most of the driving was done on a basis of two eight-hour shifts, with each shift normally completing a cycle. The time between shifts permitted the tunnel to clear after shooting without loss of working time, and also afforded time for repairs to equipment and miscellaneous work. This arrangement was an economical one as far as direct costs were concerned, with the offsetting disadvantage of higher fixed overheads due to longer elapsed time required for the job. In general, 8-ft. holes were drilled, pulling from 7 to 7½ ft. per round. The best month's progress for two-shift work was 481 ft. of 14 ft. 3-in. tunnel, or 7.8 ft. per shift.

The ventilating system was designed to provide for either blowing or exhausting through the air pipe. However, it was found that the 20-gage, 22-in.-diameter air pipe tended to collapse if used as an exhaust pipe, so the system was operated only under positive pressure. A large blower was installed at each portal, and booster fans were placed at about 4,000-ft. intervals.

Lining specifications

Particular attention was devoted to the preparation of the contract and specifications, to make them definite and explicit with emphasis on the general guiding principles rather than on details. As many uncertainties and variables were eliminated as would be consistent with the carrying out of a straight-bid contract. The hazards of tunnel construction make it to the best interest of the owner to reduce uncertainties for the contractor as far as possible, as these must be reflected in the bid.

Diamond-drill borings were made 1,400 ft. in from the Seton portal both with horizontal and inclined holes. A thorough geological report was ob-

tained, and all of these data were included with the information for bidders. Hauling to the tunnel portals was done by the company at a fixed price, and materials and electric power were also furnished to the contractor at a stated price.

A particular feature of the contract was the provision for payment of the concrete that was required to fill the volume of overbreak. The basic calculation of yardage in the lining was to be made from the prescribed lines shown on the drawings, with the actual volume of permanent timbering deducted. In addition, it was specified that concrete placed beyond the prescribed lines of excavation was to be paid for at a fixed price of $8 per cubic yard. As this figure was estimated to be about $2 less than the actual cost of the concrete in place, there remained an incentive for the contractor to reduce the overbreak in excavation as far as practicable, but the gamble involved in forecasting the amount of overbreak for bid purposes was greatly minimized.

The quantity of overbreak concrete was determined by deducting from the total amount of concrete placed in the work the amount calculated and paid for within the prescribed lines. As a basis for arriving at the total amount placed, tests were made at intervals to determine the factor for reducing batches of wet concrete to cubic yards of concrete in place. The total amount of concrete placed was then determined by counting the batches and applying the factor.

Grout was specified as one part cement to 2½ parts sand, to be paid for by the cubic yard of grout measured by volume in the grouting gun. The amount of cement required per cubic yard of grout for the specified mix and workable consistency was determined and agreed upon by trial.

Concrete lining

At the beginning of lining operations, which were carried on almost entirely from the Bridge River end, it was planned to mix the concrete on the outside, haul it into the tunnel in cars and dump it into the concrete gun. However, because of the long haul and probably in part due to the angular nature of the aggregate, it was found that the concrete packed in the cars until it had to be shoveled out. Various admixtures and additional cement were tried but although they helped to some extent, there was still considerable time lost in emptying the cars.

The procedure was changed, and the concrete was mixed in the tunnel with a 10-cu.ft. mixer kept close to the placing operations. Concrete was discharged into a hopper, which was raised by an air hoist and loaded directly into the concrete gun. The gun was mounted on a wheeled carriage and moved between mixer and forms.

QUANTITIES OF OVERBREAK CONCRETE

Section	Cu.Ft. Per Foot	Equivalent Added Thickness Around Perimeter of Section, Ft.	Ratio Overbreak to Neat Concrete, Per Cent
14 ft. 3 in. horseshoe, untimbered	0.92	0.50	83.0
14 ft. 3 in. horseshoe overall including short sections partly timbered	0.97	0.52	73.0
8 ft. 10 in. circular, fully timbered	0.04	0.03	2.4
16 ft. 0 in. circular surge chamber, about ¼ timbered	1.04	0.49	35.0
7 ft. 0 in. circular shaft about 7 per cent timbered	0.55	0.57	93.0

Length of haul did not exceed 150 ft., the mixer being moved once for each three 50-ft. sections of forms.

The amount of overbreak concrete—that is, concrete placed beyond the prescribed line of excavation, determined by batch measurement—is shown in the table for various sections of the tunnel. It should be noted that in timbered sections the prescribed line was set back to include the timber and lagging, and this applied to all permanent timbering, even where full sets were not placed. The volume of timber was deducted in computing the concrete within the prescribed line.

Grouting

The entire length of tunnel was pressure-grouted after the concrete lining was in place. Grout pipes 2 in. in diameter were left in the crown of the arch at the time of lining. These were spaced at 10-ft. intervals and located just off the center line of the arch to clear the discharge pipe of the concrete gun. Two crews were used in grouting, each with a 7-ft. grout gun of mechanical self-mixing type. The equipment was mounted on a flat car trailing a second car, which carried the immediate supply of sand and cement.

Fig. 4—Steel sets designed for a load of 5,000 lb. per sq.ft. of tunnel surface were provided for emergencies in the bad ground.

General procedure was to start grouting with one or two batches of neat cement and then progressively add sand to the mixture until it reached a proportion of 1:2 or 2½. A pressure of 60 lb. per sq.in. was normally used for refusal pressure, but this was increased to 100 lb. near the intake.

The average quantities of grout used in various sections of the tunnel were: 11,000 ft. of horseshoe or circular section (14 ft. 3 in.), generally untimbered or part timbered—0.280 cu.yd. per foot; 1,500 ft. of circular section (8 ft. 10 in.), generally timbered—0.924 cu.yd. per foot; 185 ft. of circular surge chamber (16 ft.), about one-quarter timbered—0.981 cu.yd. per foot of tunnel.

From these figures it is evident that in untimbered rock section of tunnel, even though the concrete lining is carefully placed by pneumatic methods (which are undoubtedly superior to hand methods) voids will remain between lining and rock at or near the crown of arch that are not filled with lining concrete. In the Bridge River tunnel these voids, in terms of grout (less water constituent), amounted to from 3 to 3½ cu.ft. per linear foot of tunnel. Shrinkage of the concrete accounts for some of this but is a comparatively small factor. With the grouting pressures and the amount of sand used in the mix, it is believed that no appreciable amount of grout was forced into the small seams in the rock.

In the case of the timbered sections the amount of concrete placed indicates that very little of the lining concrete found its way back of the timber lagging, even though the lagging was comparatively open. Practically all of this space behind the timbering had to be filled with grout.

Personnel

The design and supervision of the work on the Bridge River project is being carried out for the British Columbia Power Corp., Ltd., by the construction department of the British Columbia Electric Railway Co., Ltd., under the direction of E. E. Carpenter, consulting engineer, with the writer as principal assistant engineer. E. H. Hatch is construction engineer directly in charge of all field work, and L. S. McLennan is designing engineer. The contract for the construction of the entire tunnel was carried out by Pacific Engineers Limited and Henry & McFee Contracting Co., Inc., a combination of the two organizations. All concrete and grouting work was handled by the firm of Wattis, Samuels & Company under a subcontract.

All construction and operation of the generating plants of the company are under the jurisdiction of J. I. Newell, vice-president and plant manager.

Transporter Suspension Bridge to Thrill World's Fair Visitors

On an 1,850-ft. span between towers 600 ft. high a novel cable and rope suspension system will carry rope tracks at the 200-ft. level, upon which 32-passenger cars will run—To be known as the "Skyride"

By William G. Grove
Robinson & Steinman, Consulting Engineers, New York, N. Y.

JUST as the Ferris wheel, revolving passengers in a vertical plane, was the chief mechanical attraction at the Chicago World's Fair of 1893, so it is expected that the outstanding mechanical attraction of A Century of Progress at Chicago next year will be the Skyride in "rocket" cars operating in a horizontal plane, more than 200 ft. above the waters of Lake Michigan. Plans are now under way providing for the construction of this unique amusement feature, which it is expected will be in operation about a month before the opening of the exposition on June 1, 1933.

The Skyride basically is a suspension bridge in which moving cars hung from the suspension system replace the conventional roadway deck. In Europe, where bridges incorporating a moving car are fairly numerous, the term "transporter bridge" is applied to them. The best-known bridge of this type in the United States, that at Duluth, Minn., was replaced several years ago with a vertical-lift bridge. The Skyride is of much larger span than any of its predecessors. Its cars operate at a high level, while the cars of most transporter bridges are suspended with only a small clearance over the water. With towers 600 ft. high, a span of 1,850 ft. and a suspension and stiffening system composed almost entirely of wire cables and ropes, the Skyride will be a suspension bridge of note. In addition, its mechanical design is interesting, and plans contemplate architectural features as striking as the modernistic buildings that compose the exposition.

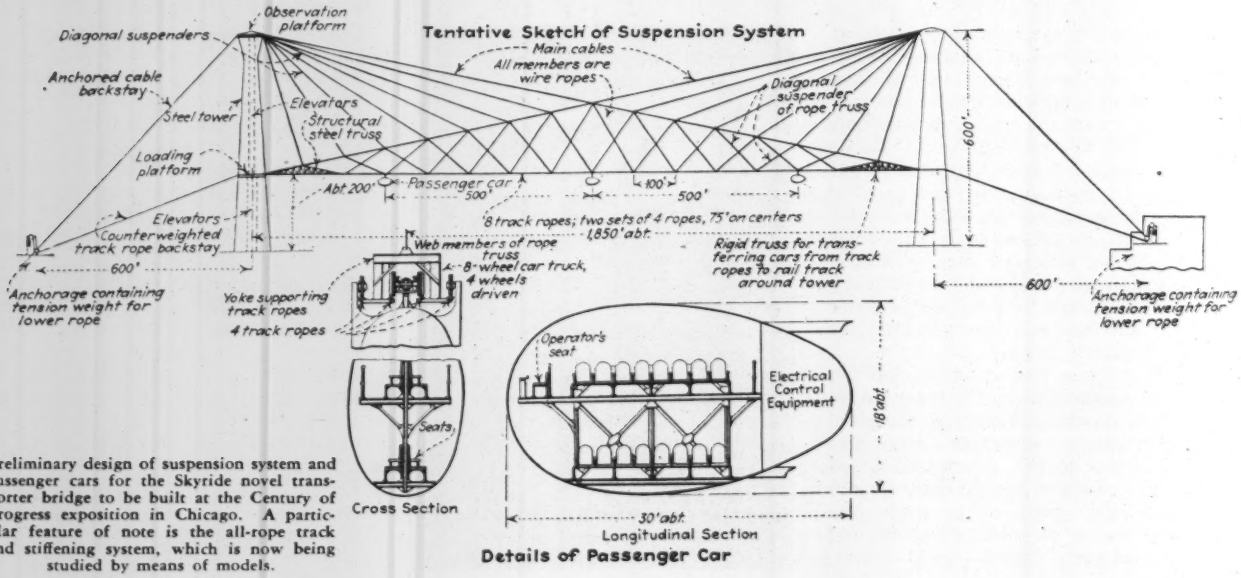
Private financing

Five companies will participate in this novel enterprise. The Great Lakes Dredge & Dock Co. will build the piers and anchorages. The structural steel will be furnished by the Inland Steel Co. and fabricated and erected by the Mississippi Valley Structural Steel Co. The Otis Elevator Co. will furnish and install the elevators in the main towers, while the cables, track ropes and cars will be furnished and erected by the John A. Roebling's Sons Co.

The exposition grounds are located in the area between the lake and Michigan Ave., extending southward from Roosevelt Road. At the upper end of this area is a lagoon crossed by a bridge on the line of 16th Street. The Skyride will be constructed at about this location, in such a way that passengers will ride in an east and west direction. The west tower and anchorage will be located immediately south of Soldier Field stadium, while the east tower and anchorage will be located near the shore line of an artificial island upon which a number of the exposition buildings have already been built. The accompanying drawing shows a tentative outline of the cable system and proposed sections of the passenger cars.

Suspension System—Instead of the cable supporting the conventional form of structural-steel stiffening trusses and floor system, the entire truss system will consist of wire ropes integral with the main cables. This wire-rope truss system will support four horizontal track ropes, each 1½ in. in diameter, at a height of 200 ft. above the lake level. In order to reduce the deflection and permit of easy riding, these track ropes will be counterweighted at each end so as to be under a high initial tension. The stiffening rope system shown is preliminary, and experiments are now being conducted at the Roebling Co.'s Trenton plant to determine the stiffest as well as the most economical form of wire-rope stiffening-truss construction. Four elevators will be provided in each tower. Two of these will convey visitors to the top of the tower, while the two others will operate between the ground and a platform at the 200-ft. level, where the cars will be loaded and unloaded. Each of the elevators will have a capacity of 32 passengers and will operate at a speed of 500 ft. per minute, making 25 complete trips per hour. The four elevators thus will provide for a peak capacity of 3,000 customers per hour.

Cars—The installation will include twelve cars, each with a capacity of 32 persons in addition to the operator. The cars will be independently operated and controlled instead of being attached to an endless hauling rope, as has been previous tramway practice. They will be double-deck, with passengers seated



Preliminary design of suspension system and passenger cars for the Skyride novel transporter bridge to be built at the Century of Progress exposition in Chicago. A particular feature of note is the all-rope track and stiffening system, which is now being studied by means of models.

back to back, as shown in the sketch, and will operate on the four track ropes. At each panel point of the rope truss a structural-steel yoke will be suspended, this yoke in turn supporting the track ropes. Each car will be suspended from an eight-wheel trolley, of which the four outer wheels will be merely supporting wheels, while the four inner wheels will be driven by means of a motor with suitable gear reduction connected to the axles. These inner wheels will have special grooves so made as to permit of sufficient clamping action on the ropes to develop the necessary tractive force required to move the car up slight grades. Experiments are in progress to determine the power required to operate the cars under the various grade, wind and weather conditions that might prevail during the period of the exposition. The car itself will be suspended from a universal joint. To prevent the passengers getting too much of a thrill due to excessive sway of the car, guides will be provided to limit the amount of this sway.

The cars will be provided with bumpers to absorb any shock caused by possible collision as the cars approach the towers. Also the entire system will be electrically interlocked, so that if for any reason one car becomes stalled out on the span, power will be cut off the following car when it reaches a predetermined distance from the stalled car. Furthermore, should anything happen to the operator, the controller will be arranged so that current will be immediately cut off from the motor and brakes will be applied to the machinery. The cars will travel at a speed of about 5 miles per hour, so as to give the passengers ample opportunity to enjoy the aerial view of Chicago as well as the exposition grounds.

Provision will be made at the tower, possibly by means of bracket-supported spur tracks, to store four cars when traffic does not require the use of the entire twelve. These storage facilities can also be used for cars that may be in need of repair.

The general method of operation will be for passengers to be taken by elevators in the towers to the platforms at the 200-ft. level. Each car will then be loaded and sent under its own power across the 1,850-ft. span to the opposite tower, the cars being kept about 500 ft. apart. As a car approaches the opposite tower it will leave the track ropes at the point where these ropes join the small truss shown in the sketch and will run on a rail attached to this truss. This rail will be inclined in such a way as to retard the movement of the car and assist in bringing it to rest. The passengers will then disembark, and the car will move on a circular rail to the opposite side of the tower, where it will be loaded with other passengers and will make the trip in the reverse direction.

Provision will be made for such passengers as have completed the trip to take the elevators to the tops of the towers, where telescopes and binoculars will be provided to view the surrounding country.

The entire Skyride project is under the general supervision of Nathaniel A. Owings, of Chicago, development supervisor of A Century of Progress. The engineering features will be under the direction of Joshua D'Esposito, consulting engineer of Chicago, and Robinson & Steinman, bridge engineers, of New York City.

Letters to the Editor

Saving Cost in Government by Private Design

Sir—The writer has read with great interest, "How Can City Costs Be Lowered?" by Clarence E. Ridley, in your issue of July 7, 1932. The National Economic League recently selected economy and efficiency in government as the most important problem of the present depression. Obviously the cost of government must be greatly reduced or the economic strength of our people will be seriously impaired. In fact it has already been impaired.

There is one waste in government which goes on continuously and which can be stopped or greatly lessened without curtailing any activity. I am convinced that the plan that I am about to suggest, if generally adopted, would reduce the cost of public works, increase the employment of engineers and architects and stimulate individual and private initiative.

We have built up in our government public-works departments (national, state, county, city) manned by staffs of engineers and architects, and these departments have been expanded to take over practically all of the public work formerly handled by engineers and architects in private practice. In the writer's opinion this is responsible for some of our increased cost in government and is not a healthy condition or one that results in the highest efficiency in design. The incentive to produce the most economical designs is largely removed by the total lack of competition under the system.

I wish to suggest that when a publicly owned building (or bridge) that is to cost more than a certain sum is to be planned by a public bureau, this bureau should employ also architects and engineers in private practice to prepare alternate plans and specifications to fulfill the same requirements. These alternate plans would then be advertised for bids together with the bureau plans. If the lowest responsible bid on the alternate plans is 10 per cent below the lowest bid on the bureau plans, the contract would be awarded on the alternate plans, and the architects and engineers employed to prepare them would be paid the usual fees for such service; otherwise only the actual out-of-pocket cost would be paid for the alternate plans. Or, if the alternate plans will not save 10 per cent of the cost, but it is desired to adopt them for other reasons, let the fee be four-fifths of the usual fee.

The writer has made repeated efforts to induce bureaus and departments to adopt such a plan of procedure for specific projects on which he knew that a substantial saving could be made, but in only one instance has he been successful in having it adopted. In that case a small highway bridge that was to cost $28,000 was built for $12,000, including the engineering cost.

WALTER H. WHEELER.

Minneapolis, Minn.,
July 12, 1932.

John B. Myers

Sir—In St. Louis, more than 40 years ago, four young men, all engineers, were closely associated in work, in pleasure and in recreation. Today the writer is the only one of the four friends left, and he here attempts to set down a few lines as a tribute to the memory of the third one of the four to go.

John B. Myers was a man of unusual honesty in business and of ready generosity to his associates and friends. He was too just and fair ever to become a rich man, except in those intangible things that usually are not classed as wealth but which bring to a man the appreciation and friendship of his fellows and that inward satisfaction that makes for placid days and peaceful nights.

He was always ready with sympathy and substantial help to those in trouble and went beyond all reasonable bounds many times for the sake of an old acquaintance.

On the day he was laid to rest (July 2, 1932), as I listened to the eulogy of the priest over his coffin, I thought that the holy man might have said of him the very same words spoken by the Great Agnostic at the burial of his brother Eben Ingersoll:

> "Were every one to whom he did some loving service to bring a blossom to his grave, he would sleep tonight beneath a wilderness of flowers."

EDWARD E. WALL.

St. Louis, Mo.,
July 20, 1932.

Engineering News-Record

EDITOR—F. E. Schmitt. EDITORIAL STAFF—V. T. Boughton, W. G. Bowman, F. W. Herring, C. S. Hill, H. W. Richardson, W. W. De Berard, N. A. Bowers, J. I. Ballard.

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AUGUST 11, 1932 Volume 109 Number 6

Proceed with Caution

TO DATE, the astonishing outburst of speculative enthusiasm which animates the markets has found no direct reflection in business or construction circles. Yet in construction as well as in commerce there are many signs, as for some weeks past, that restorational forces are counteracting the downward pressure. Contract award figures show increasing activity, and there is prospect of further advance. Favorable indications in other business fields support the hope that before winter revival may win over unemployment, especially if relief act operations are started soon. Business and construction, lacking the temperamental character of the markets, are likely to follow the prudent course of proceeding with caution, but all signs now read proceed.

Danger in Short Selling

THE PRICE CURVE of construction materials has leveled out and begun to rise, and all present indications point to further rise rather than to renewed decline. Contractors who have been bidding on a falling market are not safe in continuing to discount the future. With the funds provided by the federal relief acts carrying prevailing-wage and thirty-hour-week requirements, the wage curve also will level out and begin to climb as construction workers are taken up by the emergency highway program and the self-liquidating public works that are bound to follow the loan funds made available by the Reconstruction Finance Corporation. Prudent forethought of these probable trends of the labor and materials market appears advisable. Contractors who neglect it in estimating costs may find their condition worse instead of better as construction improves. Any bidder who persists in selling short is, according to present indications, taking an unwise risk.

Take Stock!

WITH the likelihood that the Emergency Relief and Construction Act will initiate a resumption of public works in large volume, inventories of construction equipment are in order. Three years of curtailed construction have not improved the idle plant, and meantime the decreased production of equipment has put the wits of manufacturers to work bettering old machines and inventing new ones. Even where old equipment has been maintained at the peak of condition, obsolescence has taken its toll; but maintenance has only rarely approached perfection, and in consequence most old machines lack their original speed and endurance. When contractors again have to meet keen competition, the bidder whose plant is able to assure lowest cost will have the best chance. Every contractor with old plant can spend some profitable time in contrasting it unit by unit with the machines that the market now offers; he will at least learn the equipment handicap that he has to overcome by greater skill in his operations.

Signs and Dangers

ARE lane-marking and pavement signs too complex? The Massachusetts highway department has concluded that they are. It therefore proposes to limit signs and markings to the few locations and simple forms described in this issue. All warning marks are to be taken off the pavement, and traffic lanes are to be indicated only at points of hazard, such as curves and some intersections. Complex marking, it is argued, merely delays traffic and increases the chance of accident. The argument is worth consideration, for recent years have seen steady multiplication of highway sign and symbols. If we are approaching the point of surfeit, disregard and increased danger, it is well to recognize the fact in time.

Hard Facts on Hand Labor

ACCUMULATING records of road construction by hand labor indicate that displacement of machine methods has to be paid for by higher costs. In Michigan, where nearly $12,000,000 has been spent in hand-labor operations, the practice is reported as "less economical," though the highway department took care to select operations best suited to manual work. Now the state of Washington reports highway bid prices to be 38 to 70 per cent higher for hand than for machine work; the spread may be exceptional, or the jobs may not have been best suited to hand performance or else were badly organized, but the difference remains. If the full spread of employment were computed, it is doubtful if either Michigan or Washington increased employment by discarding machine methods. The Congress and legislatures in mistaken zeal may insist on out-dated procedure, but engineers should keep clearly in mind that it is a step backward. To keep the retrogression from being disastrous will call for all the skill and invention that highway engineers can bring to bear in so choosing and organizing operations that hand labor will be efficient.

Tension Stiffening Systems

BECAUSE of the prominence of suspension bridges in recent construction history it is peculiarly interesting that a suspension bridge is to be the support of the novel device which, as successor to the Ferris wheel of forty years ago, is to be the popular attraction at the 1933 exposition in Chicago. But it is a bridge of a new kind, for in place of a stiffening truss it will depend on a tension system of ropes to give rigidity to the track members, which themselves are ropes. The design is highly suggestive, quite beyond its present application. Ropes were used as auxiliary stiffening devices in the Brooklyn Bridge, a structure which also has a stiffening truss; and though often criticized as superfluous and probably harmful, they are still in place and are giving good service. However, the simple arrangement of diagonal stay ropes there used is carried to wholly new levels in the Chicago design by secondary trussing of remarkable character. Model tests have demonstrated the effectiveness of the system, it is reported. Nevertheless, its large-scale working is sure to yield valuable experiences.

Self-Liquidating Public Works

THE MOST important step yet taken toward the rebuilding of the nation's economy is the provision by Congress of a $1,500,000,000 credit fund for self-liquidating public projects. Its objective, as we have already emphasized, is not to build public works but to create employment and stimulate business activity. If this objective is to be attained, the potentialities of the great credit fund must be utilized to the full.

Carrying the new credit provisions into application involves many difficulties, however. Serious problems are sure to appear in devising the necessary administrative measures, and grave questions of public policy will be involved. But these difficulties can be overcome. The responsibility rests with the states and cities of the country.

At the outset much depends upon the general attitude taken by the R.F.C. as to the proper function of the funds at its disposal. If loans are to be made grudgingly and with the same rigid requirements as would be set by the commercial banking fraternity, there will be small improvement over the conditions of the past few years. Further, upon this general attitude will largely depend the stand taken by the corporation upon the many complicated questions of eligibility that are sure to arise.

Even with those improvements whose general nature is obviously of the self-supporting type, debatable questions arise that the R.F.C. must answer promptly. What is to be the interpretation of the requirement that the cost of construction must be "returnable within a reasonable time?" In the case of a new toll tunnel, must interest and amortization charges be earned during the first year or so of the life of the structure? Good business sense would recognize that in all such structures there is a preliminary period of deficient earnings.

What will the R.F.C. conceive to be the meaning of the term "project"? In the case of a water-supply system, for example, is it to mean the system as a whole or merely the improvements or additions for whose financing a loan application is being made? The cost of constructing an elevated tank to improve pressure conditions might be easily returnable out of the waterworks revenue, but the tank in itself would not measurably increase that revenue.

How is supplemental income to be regarded? Must tolls and charges on the individual new structure be capable of returning its cost completely, or may they be supplemented by revenue of the same character from a neighboring structure? Specifically, can the earnings of an existing toll bridge be pledged to aid in the support of a new one to be financed out of R.F.C. funds? Improvements of another type might be capable of paying for themselves out of resultant economies, such as savings in rentals. Would such improvements be eligible for credit? The whole field of special-assessment improvements is open to debate. Are special assessments for benefits to be considered taxes? Some sort of codification of eligibility rules by the R.F.C. administrators seems essential, but until such formulation is forthcoming the local administrators can go far in preparing their plans in accordance with what is clearly the purport of the relief act.

Questions of equal magnitude confront local administrators in their attempt to cooperate in this great effort toward recovery. If the relief act is to have any appreciable measure of success, modification in the method of financing many public improvements will be unavoidable. Confining new projects to the type that cities are accustomed to make self-supporting will not go far enough in utilizing the relief act's full potentialities. Improvement of the type that are now ordinarily supported by taxation must be made self-liquidating.

Right here arises a serious problem of public policy. Public authorities may be loath to reverse the trend of the past generation and to begin to set up financial structures based on tolls. But such a development need not be considered permanent; rather it is to be viewed as an emergency measure aimed to realize the full effect of the relief act, and when the emergency is over the old methods of financing on tax support can be restored.

What facilities that are now ordinarily tax-supported can conceivably be put on a revenue basis, and what new problems will thereby be involved? Long-span bridges immediately come to mind as capable of producing revenue. But it becomes obvious at once that a near-by free bridge would be a formidable competitor of a new toll structure. Accordingly, such possible competitors would also have to be put on a toll basis.

Waterworks systems now are largely self-supporting in theory, but in all likelihood they will have to be made so in fact to the satisfaction of the R.F.C. if they are to be eligible for borrowing from the relief-act credit fund. This will require that their accounting methods be made more rigid than is now usual, and their revenues segregated more carefully from general funds. Revenues will have to be made sufficient to carry all capital costs in addition to the cost of operation. But, quite properly, the waterworks might claim specific payments from the city corporation for fire-protection service and water consumed by public institutions. Such financial arrangements are sound at any time, and the more sound if needed to meet the requirements of the R.F.C.

Sewerage systems can be put on a self-supporting basis with a great deal of propriety. There are now twenty states in which cities can legally support their sewerage system on a service charge basis. It is true that there are large items of cost involved in a sewerage system that are of general community benefit and cannot properly be charged for on a utility basis. The proportion of the cost represented by provisions for stormwater drainage, for example, could possibly be covered on the same basis as that suggested for the fire-protection service rendered by water systems. Sewage-treatment works present independent questions, but none beyond solution.

There are many other services now supported by taxation that can be made self-supporting. Garbage disposal is one; service charges for this are not difficult to arrange. Certain park facilities might be made self-liquidating by being placed upon a free basis. But in many cases these developments toward self-support for municipal services must await legislative action. Accordingly, they depend absolutely upon the support of public opinion.

Enlisting popular support is the most important problem of the local authorities, in short. It is here that the engineering profession has an obvious duty to perform. The ultimate initiative must come from public-works engineers, for they are the only ones who are sufficiently familiar with the characteristics of public improvements to be able to appraise their true value. Without the enlightened energy of the public-works engineer the support of local citizens will be difficult to obtain, and the provisions made by Congress for national economic recovery will be futile.

NEWS OF THE WEEK

Engineering Board Named to Pass on Loans by R.F.C.

A BOARD OF ENGINEERS to pass upon the engineering adequacy of projects for which applications for loans are made to the Reconstruction Finance Corporation has been named by President Hoover. It consists of six members and is headed by Charles D. Marx, professor emeritus of civil engineering at Leland Stanford University. Other members of the board are John F. Coleman, New Orleans; John Lyle Harrington, Kansas City; John H. Gregory, Baltimore; Herbert G. Moulton, New York; and Major General Lytle Brown, Chief of Engineers, U. S. Army, ex-officio. The chairman of the board was the President's own selection. The other members were selected from a list submitted by American Engineering Council.

The board was selected so that each of the major geographical divisions of the country might have representation. Consideration also was given the experience of the members along the lines that most of the applications for loans are expected to cover.

Prof. Marx was selected to represent the West and because of his specialized knowledge of municipal improvements. Mr. Coleman represents the South and has a specialized knowledge of port development. Mr. Harrington represents the Mid-Continent territory and provides expert knowledge of the toll-bridge situation. Mr. Gregory represents the East and was chosen because of his intimate knowledge of water works and sewage disposal. Mr. Moulton, originally from Idaho, represents the Northwest and was chosen largely because of his knowledge of the financial side of engineering projects.

Personnel of the board

Charles D. Marx, chairman of the board, occupied the chair of civil engineering at Leland Stanford from its establishment in 1891 until his retirement in 1923. He is a graduate of Cornell and of Karlsruhe, was in railroad and river work for a few years after graduation and taught at Cornell and the University of Wisconsin before going to California. He has done consulting work frequently and for some time served as chairman of the state water commission of California. He is an honorary member and past-president of the American Society of Civil Engineers.

John F. Coleman has been in consulting engineering practice in New Orleans since 1900, specializing in river and port work, land reclamation and railroad and utility valuation. He began his engineering career as a rodman on railroad location in Guatemala in 1884 and subsequently was on railroad construction and river work in this country. For a time he was principal assistant city engineer in New Orleans. He also is a past-president of the American Society of Civil Engineers.

John H. Gregory is a consulting engineer in Baltimore, also professor of sanitary engineering at Johns Hopkins University. He is a graduate of Massachusetts Institute of Technology and has been in sanitary engineering work since his graduation in 1895, first in Boston with the Metropolitan Sewerage Commission and the Metropolitan Water Board and subsequently in private practice in New York and Baltimore.

John Lyle Harrington is a member of the consulting engineering firm of Harrington & Cortelyou, Kansas City. He has been in bridge work, both in the structural shops and as designing engineer, much of the time since his graduation from the University of Kansas in 1895. For the past twenty-five years he has been in consulting and designing practice successively as member of the firms of Waddell & Harrington; Harrington, Howard & Ash; and Harrington & Cortelyou. He is a past-president of the American Society of Mechanical Engineers.

Herbert G. Moulton is a consulting engineer in New York who specializes in reports for banking interests on railroad and mining operations and finance. He is a graduate of the University of Oregon in mining. During the war he served with the War Industries Board and from 1915 to 1920 was adviser to the Public Service Commission of New York on timbering in the New York subways.

Lytle Brown has been Chief of Engineers, U. S. Army since 1929, when he was appointed over the heads of several senior officers on the grounds that a man of unusual executive ability was needed to take charge of the large flood-control operations then being undertaken by the corps. He is a graduate of West Point and has had wide experience in federal government work both in the engineer corps and on the general staff of the army.

R.F.C. Receives Applications for Self-Liquidating Project Loans

Eligibility rules are not yet published but applications for loans for self-supporting improvements accumulate from cities and special districts—Highway work gets under way

NO FORMULATION of eligibility rules for loans for self-liquidating projects has yet been made public by the Reconstruction Finance Corporation. Formal applications for loans have been made so far in only a few instances, but public agencies throughout the country are preparing to ask for funds to carry out construction of self-liquidating public works. Highway construction, aided by the $120,000,000 additional federal aid fund, has been getting under way quickly.

Vehicle tunnel for New York

Notice was filed with the R.F.C. on August 4 that an application will be made for a loan of $75,000,000 by the Port of New York Authority for the proposed mid-town Hudson Tunnel. Formal application will be deferred until after conferences of a special committee of the Port Authority with a committee of the R.F.C., when terms and conditions of the loan will be considered.

The proposed tunnel is extended under the Hudson river between North Bergen and Weehawken, N. J., and a point in the vicinity of West 38th St., New York City. It is estimated that $20,000,000 will be required for each of the first three years to carry on the work, $10,000,000 for the fourth year and $5,000,000 for the fifth year. Based upon the latest plan at prices now prevailing, the cost of construction and initial equipment required for operation is placed at $43,700,000; contingencies and engineering, $7,500,000; real estate and rights of way, $17,000,000; interest during construction, $6,800,000. Preliminary planning and design work have been completed and it is estimated that the new tunnel could be opened to traffic in the fall of 1937.

A loan of $160,000,000 has been formally applied for by the National Gulf-Atlantic Ship Canal Corp. to build a proposed ship canal across Florida. A clause in the application states that the canal would become government property after it has paid for itself.

The preparation of plans and specifications for a proposed $1,861,000 improvement program for the Seattle water department has been made possible by an appropriation of $10,000 by the city council. The city plans to borrow the construction money from the R.F.C. on water department bonds. The principal projects involved in the improvement program are: replacement of the Cedar River pipe line, which is greatly deteriorated; slope dressing and drainage of the ground surrounding the new West Seattle reservoir; replacing with steel the old wood main on Alki Ave.; installing a pumping plant and main to supply a large area north of the city; constructing a 60-million-gallon basin at Maple Leaf reservoir, doubling the present capacity; and reconstructing with concrete the wooden portion of the intake dam at Landsburg.

Municipal improvements in Buffalo

Buffalo, N. Y., is also preparing to apply for federal financial aid for municipal construction. The projects for which loans are to be asked from the R.F.C. include: two bridges to connect Grand Island with Niagara Falls and Buffalo, to cost about $7,000,000; an addition to the Buffalo incinerator; a sewage disposal plant for Buffalo; a sewage disposal plant for Lack-

awanna, to cost about $1,000,000; and a number of highway projects.

Citizens of Key West, Fla., have discussed with members of the R.F.C. application for a loan of $12,000,000 for the purpose of completing the Overseas Highway from Key West to the Florida mainland. This proposed highway is intended to provide for motor traffic from all sections of the country to Havana via Key West. Cuban officials are likewise evincing increasing interest in the project.

To further their plans for borrowing $32,000,000 from the R.F.C. for the construction of a transmission line to Hoover Dam, Los Angeles, Calif., board of water and power commissioners has been authorized by the city council to employ Joseph Scott and William Gibbs McAdoo as special counsel.

The Huron Road Hospital, Cleveland, has applied for a loan from the Reconstruction Finance Corporation in order to complete its new hospital in Cleveland Heights. The hospital now occupies two old buildings on Euclid Avenue at 88th St. The completed project will cost about $950,000, and is now two-thirds complete.

Pasadena, Calif., has asked for a loan of $6,500,000 for construction of the San Gabriel water project.

American Engineering Council has sent out communications to each of its state committees throughout the country asking them to work with government agencies in recommending loans for self-liquidating projects and in obtaining employment for engineers in public works construction.

State road contracts have been advertised in Utah totaling $172,000, making use of the state's share of emergency funds for highway projects. The contracts advertised, include 14.4 mi. of gravel construction in Millard county; graveling 9 mi. in Beaver county; oiling in Beaver and Millard counties on 25.5 mi., and 3 mi. of road in Price Canyon, Carbon county, to be surfaced with either bituminous concrete or rock asphalt.

Formal application for $15,500,000 for additional flood control works in the lower Mississippi Valley, as authorized by the relief act, has been made to the War and Treasury departments by Major General Lytle Brown, chief of army engineers, on the recommendation of the president of the Mississippi River Commission. The probable apportionment will be $6,500,000 to the Memphis District; $1,500,000 to the Vicksburg District; $3,500,000 to the New Orleans District; and $4,000,000 for bank protection, contraction work and dredging.

Three direct relief loans aggregating $5,652,662 have been authorized by the R.F.C. The corporation has extended aid to the state of Illinois, the city of Detroit, and four counties in Ohio. Illinois was granted a loan of $3,000,000, Detroit was granted $1,800,000 and four counties in Ohio $852,662. Louisiana has applied for a loan of $12,000,000 for direct relief; Washington, $35,000,000; Montana, $1,000,000; New Mexico, $1,250,000.

Lone Bid Rejected for San Gabriel Dam No. 1

Los Angeles County Flood Control District on Aug. 8 rejected the single bid it had received for building San Gabriel Dam No. 1, from Constructors, Inc., and announced that the job would be re-advertised.

Two proposals will be considered when the contract is re-advertised. One will be as before, and the other will be an alternate, permitting separate bids on excavation, tunnels, rockfill, concrete, grouting and placing steel.

Combining under the firm name of Constructors, Inc., ten firms on Aug. 1, had submitted to the county board of supervisors the only bid received for construction of dam No. 1. The company bid on two proposals as follows:

No. 1—Calling for surfacing the water side of the dam with concrete by the gunite process, $11,270,043.

No. 2—Calling for surfacing with concrete by pouring, $11,315,868.

Estimates of E. C. Eaton, chief engineer of the county flood control district, were $9,743,380.

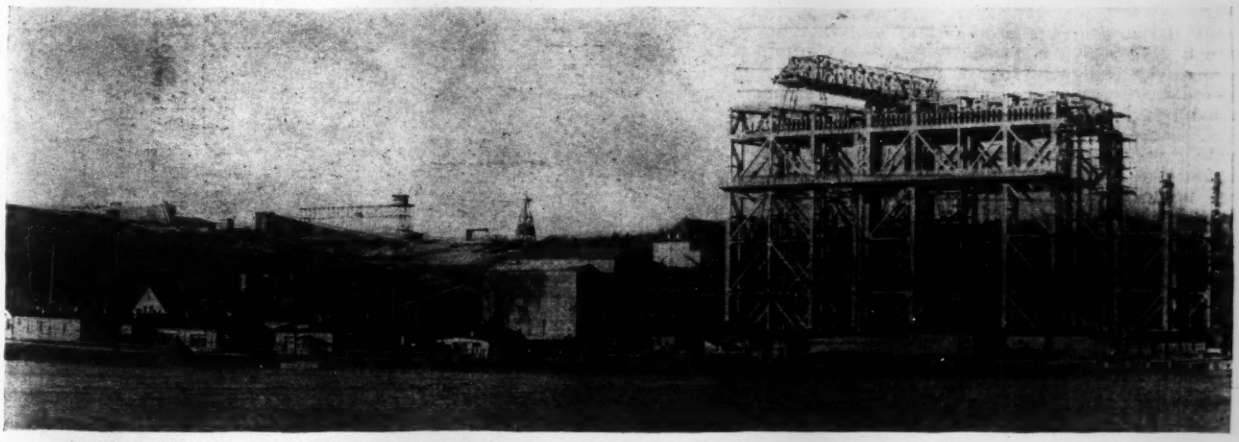
Officers of the bidding concern were announced as Arthur Bent, president; H. J. Kaiser, vice-president; L. E. Dixon, general manager; K. K. Bechtel, secretary, and G. G. Sherwood, treasurer.

California Special Water Session Will Probably Not Be Called

Plans for a special session of the California legislature to consider the state water problem are indefinite. There are indications that there will be no call because of opposition developing among the members. Governor Rolph had planned to have a program of legislation outlined and unofficially approved so that only a brief special session would be required to pass the measures to be put before the citizens at the general election in November. However, a final telegraphic canvass of the members of the legislature finds a decided sentiment against the calling of the session, either because of its cost or actual disagreement with the water program as now outlined. A further factor is the growing demand to open the special meeting to questions of unemployment, taxation and general matters, with prospects for a protracted session, whereas the original plan was to restrict the session to a programmed consideration of the water problem only.

A public meeting was called by the governor in Oakland, Calif., July 11 as a means of sounding out public opinion prior to calling the session. At the gathering, attended by several hundred citizens and members of the legislature, the proposed legislation was outlined by members of the California Water Resources Commission and the Joint Legislative Water Committee. Briefly, the plan called for approval by the legislature of a constitutional amendment relating "to the control, development and utilization of the water resources," an act creating a permanent California Water Resources Commission and a bond issue of $150,000,000 to construct the central valley project including the Kennett dam, San Joaquin pumping system and the Friant dam. These three proposals, if approved by the special session, would have been put on the ballot at the fall election.

The meeting showed distinct differences of opinion, not only on the necessity of considering the plan at an immediate special session, but as to acceptance of the water plan itself. Opposition was particularly noticeable from speakers representing the Sacramento Valley and a lack



LARGE SHIP ELEVATOR BUILT FOR BERLIN-STETTIN CANAL

In the reconstruction of the canal system between Berlin and Stettin, on the Baltic Sea, there is now under construction a marine elevator to raise barges of 1,000-ton capacity through a vertical distance of 36 meters (118 ft.). This mechanical lift replaces a system of four locks in the existing canal. The lifting operation in the new elevator will require only five minutes. The elevator in its raised position will be connected with the canal proper, shown at the left, by an aqueduct.

of interest was apparent from the southern part of the state. However, the meeting voiced a sentiment in favor of calling a special session for exclusive consideration of the water problem and the Governor announced it would be called. The telegraphic poll of the legislators the following day developed the opposition which makes the call uncertain. The proposed constitutional amendment would require a two-thirds majority and replies from state senators indicates passage would be impossible.

Highway Congress to Replace Road Show Next Year

A highway and building congress will be substituted for the convention and equipment exhibition of the American Road Builders Association which have been a notable show feature of the roadbuilding industry for many years. The congress will be held in Detroit, Mich., Jan. 16 to 23, 1933. Ten associations will join with the American Road Builders Association in simultaneous conventions and a joint exhibition of equipment and materials.

The preliminary announcement states that the program provides for concurrent meetings of the several individual organizations during the first three days of the congress, to be followed by a joint session which will map a coordinated procedure for the entire industry and allied enterprises. The associations holding meetings will be the Construction League of the United States; American Road Builders' Association; Associated General Contractors of America; Motor Truck Executives Association; Asphalt Institute; National Crushed Stone Association; National Paving Brick Association; National Ready Mixed Concrete Association; National Sand and Gravel Association, and Portland Cement Association.

Welland Canal Formally Opened

Formal opening of the new Welland ship canal connecting Lake Erie and Lake Ontario took place on Aug. 6. The canal has been in service for several months. It was opened to down traffic in September, 1930, and soon after the opening of navigation last year the remainder was put into service and completion of those parts which could not be completed until the old canal was abandoned was begun.

The new canal has a depth sufficient for vessels of 25-ft. draft, thus opening Lake Ontario to the large vessels operating in the upper lakes. The old canal accommodated vessels of 14-ft. draft and had 26 locks in its 26 miles of length. The new canal is somewhat shorter, and has only 7 lift locks and one guard lock. Each lift lock overcomes a total height of 46½ ft., the highest in the world. Lock chambers are 80 ft. wide and have a usable length of 820 ft. A depth over the sills of 30 ft. has been provided so that the canal can be deepened in the future without change in the lock chambers.

Work on the canal was begun before the war, was discontinued in 1916 and was not taken up actively again until 1921. The total cost has been about $120,000,000. A detailed description was given in *Engineering News-Record*, Sept. 4, 1930, p. 356.

Ohio Sues Columbus Gas Company for Damage to Building

The state of Ohio, through its attorney general, Gilbert Bettman, on August 4 filed suit for $1,100,000 damages in Franklin County courts at Columbus, Ohio, against the Columbus Gas & Fuel Co., natural gas distributor. The petition charges the gas company with liability for the explosion which caused serious damage to the nearly completed state office building in Columbus, April 14.

The suit is based on the grounds that a 16-in. gas main installed on Scioto Boulevard, adjoining the state building, was improperly installed, developed a leak and caused gas to form pockets in the state building, following which the blast occurred. Complaint is made that the gas company in constructing this main laid a single pipe without any ventilation, instead of constructing a line consisting of an inner and an outer pipe, the latter for ventilating purposes, through additional pipes placed at intervals of 1,000 ft. as required by the company's franchise, granted Feb. 4, 1889.

Concerning claims against other parties, Bettman stated that, "In proceeding against the Columbus Gas & Fuel Co. the state does not relinquish claims which it has against the contractors upon the building for delivering to it of a completed structure."

The Struck Construction Co., Cincinnati, is the general contractor. The building now is being rehabilitated and brought to completion. Following refusal of the contractors to proceed without additional funds, the Ohio legislature in a special session provided financial support for the completion, without prejudice to the state's claims.

"As to these claims," Bettman stated, "I feel that their determination may be held in abeyance until the building is completed, which will afford an opportunity to determine the liability of the gas company.

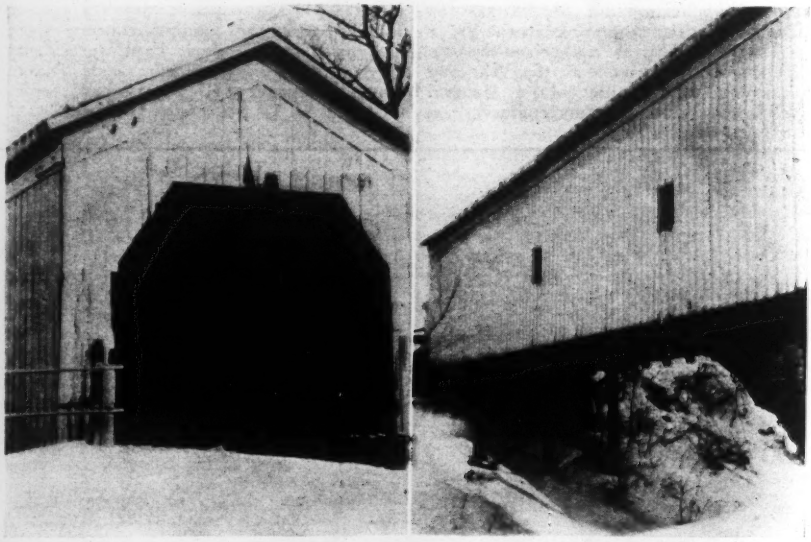
"Of course, in the event of recovery against the company to that extent the contractors will be absolved."

On August 6, the gas company won its legal battle instituted several weeks ago to obtain possession of a section of the welded pipe in front of the building. It was this section of pipe which the state declared was improperly installed.

Judge Henry L. Scarlett of the Franklin County courts ruled that the gas company should have the pipe, but that the state should have "full opportunity for inspection of the welding during the tensile-strength tests" which are to be made by the gas company. The judge's decision was on a suit by the gas company, seeking to enjoin the state office building commission from interfering with removal of the pipe as reported in *ENR*, July 21, p. 90.

Filtration Plant to Treat Well or River Water

Construction of a 3-m.g.d. filtration plant, started last September, has now been completed at Peru, Ind. The plant which is paid for from earnings of the municipal light and power plant, was dedicated July 31. It is designed to remove iron from well water or turbidity, tastes and odors from the Wabash River water with arrangements for future softening. The treatment consists of mechanical aera-



OLD TIMBER TRUSS BRIDGE TO BE PRESERVED

A covered wooden truss highway bridge over the Zumbro River at Zumbrota, Minn., is to be replaced with a modern structure, but will be moved intact to the city park and preserved as the last bridge of its kind in Minnesota. This bridge, 116 ft. long and with a 15-ft. roadway, was built in 1869 by the Strafford Western Immigration Co. at a cost of about $5,800. Its timbers are of white oak, put together with oak pins. It is in good condition, but is not strong enough for modern loading and is too narrow to accommodate the present traffic. The removal of the old bridge and construction of the new one is being done by the highway department.

tion; 30-min. chemical mixing, 4 hours settling in two 56-ft. square settling basins, one equipped with a mechanical scraper; four 14x19-ft. gravity sand filters provided with block underdrains and hydraulically controlled operation. Pumping equipment consists of one 2,000- and one 1,000-g.p.m. motor-driven pumps and one 1,000-g.p.m. pump with motor and gasoline engine drive. A complete laboratory for chemical and bacteriological analyses is provided. Charles Brossman was the consulting engineer and John W. Horan, city engineer. C. J. Burke and Burnip Construction Co. Inc. were the contractors.

Washington Builders Advocate Wage Cuts in Building Trades

The Building Trades Employers Association of Washington, D. C., has petitioned the federal government, through the Secretary of Labor, to effect through the operation of the Bacon-Davis prevailing wage law a reduction of wage rates paid to workers in the building trades in Washington, D. C. The petition asks that all trades with a present wage scale of $10 per day or less to be reduced a minimum of 10 per cent, and those over $10 a minimum of 14 per cent.

Ohio Considers Modification of Highway Financing

A special session of the Ohio legislature, planned for early September, will have presented to it by Governor White a proposal to authorize a portion of the state's gasoline tax receipts for payment of assessments on roads built before the state took over the highway system.

The plan is permissible under the state's gasoline tax law, the governor stated, and would result in payment of a "debt already contracted which we morally owe in connection with highway construction." Under a system in force a number of years ago, many citizens whose property adjoins roads built before the highway system became a state matter, are still paying assessments.

Federal Building Program Approaches Peak

The building program of the federal government, speeded up a year and a half ago by Congress to aid employment, will reach its peak of activity during the current month. With 583 projects calling for $388,963,542 either completed or under contract, almost three-quarters of the program is either finished or under way. Only 233 projects involving $107,500,400 have yet to be contracted for.

Efforts are being made in the construction division of the Treasury to bring to the contract stage the $100,000,000 voted to the building program as part of the emergency relief and construction act before next spring.

Although the peak in contract awards has already been reached, payment under existing contracts are lagging and will be at their height later. Of the 833 projects for which Congress has made definite appropriations, 216 had been completed by the end of July.

Thirty-Hour Week Adopted For Wisconsin Public Works

All public works construction under the supervision of the state of Wisconsin will be operated on a five-day week and six-hour day basis. An announcement to this effect has been made by the State Unemployment Commission. This rule will apply not only to highway work financed by federal funds but to all construction work paid for by state funds whether under contract or by day labor.

The commission has taken the stand that the best way to increase the employment of labor on public works construction is to perform the maximum amount of such labor by hand rather than by machine. In contracts awarded by the commission this policy has been followed, including the unloading by hand of all stone, gravel and sand used in concrete construction.

New Suspension Bridge Planned for Long Island Park System

Approval has been given by the War Department to the plans of the Long Island (N. Y.) State Park Commission for a suspension bridge across Great South Bay between Conklin Point and Captree Island. The bridge is to form part of the park commission's project of linking the Ocean Boulevard, running eastward from Jones Beach along the outer bar on the southern shore of Long Island, with the Southern State and proposed Sagtikos Parkways.

The project is estimated to cost about $3,500,000 and will involve a causeway 7½ miles long of which the suspension bridge is the central unit. The bridge will have a horizontal clearance of 820 ft. and a vertical clearance of 64 ft. The trestle approaches will be 3,200 ft. long with horizontal clearances of 45 ft. and vertical clearances varying from 25 to 35 ft. The plans call for a roadway 60 ft. wide.

Consideration was first given to the construction of a trestle bridge with a draw span but it was concluded that such a bridge could not be made architecturally satisfactory. A further objection was that the operation of the draw span would interrupt the motor traffic.

Conference on Construction Planned for Next Fall

At the meeting of the executive committee of the National Conference on Construction, June 27th, decision was reached to hold a general assembly of the Conference next fall. Invitations will be extended to about 15 groups. In addition to the architects, engineers, general and subcontracting groups, material supply dealers, realtors, investment and mortgage banking and surety groups will be invited. The Conference was organized in 1931 by leaders in the various branches of the construction industry to serve primarily as a service group for the various trade associations in the construction field.

At the recent meeting progress was reported by various committees including those on statistics, on waste, on realty finance, on local organizations and on economic relationships. The latter committee is considering such subjects as bid peddling, pre-qualification, etc. The selection of a president of the Conference was left in the hands of a committee. H. J. Kaiser, president, Associated General Contractors of America, and Willard Chevalier, publishing director, *Engineering News-Record*, were added to the executive committee. J. A. Ehlers, with offices in Washington, D. C., is technical director.

Contractors' License Law Sought in Ontario

Legislation to license contractors operating in the province is likely to be sought at the next session of the Ontario Legislature. A committee headed by W. B. Sullivan, President of the Toronto Builders' Exchange, and including representatives of the Ontario Builders' and Supply Dealer's Association, the Ontario Branch of the Canadian Construction Association, the Ontario Association of Architects, the Ontario Branch of the Engineering Institute of Canada and the Toronto Home Builders' Association is at present engaged in drafting a bill to be submitted to the legislature.

Regulations combining features of the licensing law now in force in the State of California, and of a local measure operating in Boston are understood to be considered most satisfactory for Ontario.

Obituary

W. D. Carter, 60 former superintendent of highways, Town of Wendon, N. Y. died at his home in Wendon, N. Y., on August 3.

William P. Beightler, aged 64, city engineer of Marysville, Ohio, and resident state engineer of the Ohio Department for Union County, died Aug. 5 at Mt. Carmel Hospital, Columbus.

John A. McLean, until early this year city engineer of Crookston, Minn., was killed July 30 by an employee of the city who had been dismissed two years ago. The murderer immediately committed suicide. Mr. McLean was 51 years old, a civil and mining engineer. He was graduated from the University of North Dakota in 1905 and became city engineer in 1920. He was an associate member of the American Society of Civil Engineers.

Henry C. Allen, former city engineer of Syracuse, N. Y., died at his home in Syracuse on August 5 at the age of 69. He had been ill since early last spring. Mr. Allen served as city engineer 1890-95 and again in 1907. During 1914-15 he was chief engineer of the Syracuse Grade Crossing Commission. He returned to the city engineers office again in 1916 serving until 1921 when he opened an office and engaged as consulting engineer.

George A. Posey, county surveyor of Alameda County, Calif., killed himself in his Oakland home August 30. Sensitiveness over a grand jury recommendation of an investigation into his office is given as the reason for his act although there are no indications of any irregularities in connection with work he did for subdividers engaged in a "free lot" racket. Mr. Posey was chief engineer for the pre-cast, subaqueous tube built from Oakland to Alameda, which was officially named the Posey Tube in his honor.

Engineering Contracts and Capital

HEAVY ENGINEERING construction contracts reported for the past week total $21,858,000, though few single large contracts were awarded. The weekly average for contract lettings for this month is $25,015,000, as compared with a weekly average of $30,166,000 for July and $37,266,000 for August of last year. Cumulative contracts to date total $765,813,000. Corresponding figures for last year were $1,713,709,000. Two of the largest contracts let last week were for a telephone exchange in Philadelphia, to cost $2,600,000 and state highway work in Texas aggregating $2,043,000.

Although public capital issues for the week were low, heavy private issues brought the week's total up to $35,800,000. Cumulative capital issues to date this year total $553,000,000, as compared with $1,652,000,000 for a similar period last year.

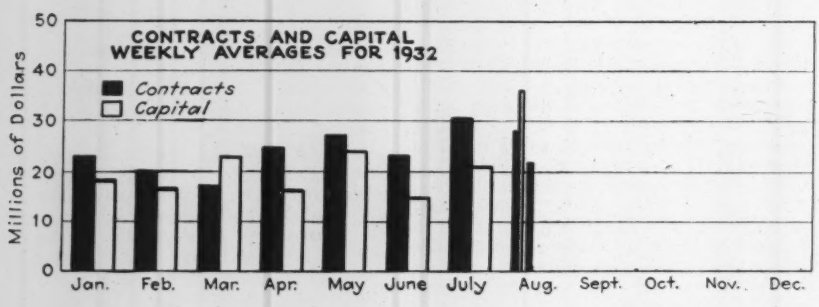
More Cement Increases—Evidently encouraged by recent advances in the midwestern and eastern marketing areas, cement manufacturers in the Birmingham district have announced an increase of 40c. per bbl. While at present only three of the six mills in the district are in operation, loans from the Reconstruction Finance Corp. to some of the states in the territory are expected to materialize and release contracts that have been temporarily held up for lack of funds.

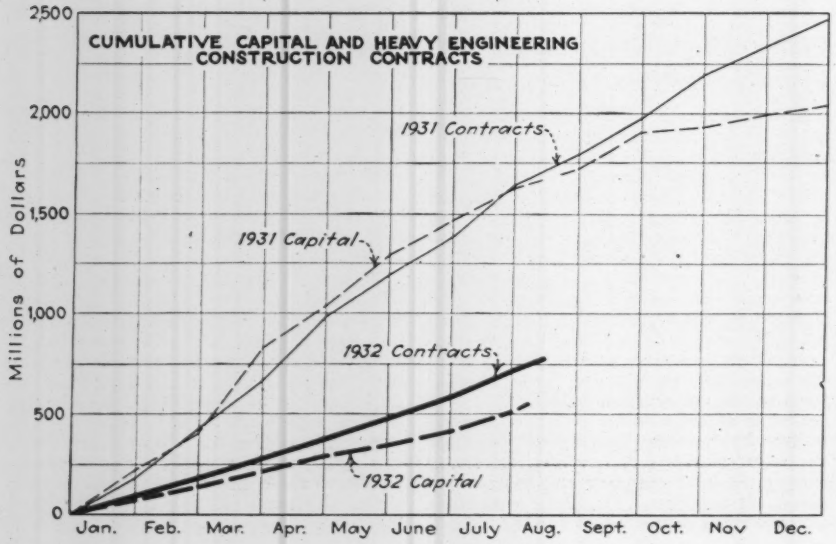
Steel Gains Expected—While production of steel fell off slightly from the 16-per cent capacity output maintained for the past three weeks, a gain is looked for towards the last of the month. Some mills are holding off certain rollings to accumulate tonnage for more economical operation later on in the month.

Marseilles Dam Contract Sublet—Stevens Bros., Miller Hutchinson Co., New Orleans, who hold the contract for the construction of the Marseilles dam on the Illinois waterway, have sublet the work to the Congress Construction Co., Chicago, following labor trouble last month culminating in a riot in which one man was killed. According to reports, the Congress Co. will use union labor and raise the rate for common labor from 35c. to 40c. per hour.

Test Contract Cancellations—A test case to determine the right of the city of Philadelphia to cancel construction contracts aggregating more than $2,000,000 will be heard in September. Mayor Moore is endeavoring to revoke many contracts let in the closing days of former Mayor Mackey's administration. The test suit is being brought by the Frederick Snare Corp., one of the affected contractors.

St. Louis Strike Settled—A strike of union employees of the General Material Co., which had tied up 52 building and paving concerns in St. Louis and vicinity since May 16, was settled Aug. 4. Considerable violence marked the strike, two persons being killed.





Contracts and Capital

CONSTRUCTION
(Thousands of Dollars)

	Weekly Average Aug., 1931	Weekly Average Aug., 1932	Week Aug. 11, 1932
Federal government	$2,710	$6,132	$5,521
State and municipal	17,904	14,488	11,033
Total public	$20,614	$20,620	$16,554
Total private	16,612	4,392	5,304
Week's total	$37,226	$25,012	$21,858

Cumulative, Jan. 1 to date:
1931 construction $1,713,709
1932 construction 765,813
Decrease, 55 per cent.

NEW PRODUCTIVE CAPITAL
(Thousands of Dollars)

	Weekly Average Aug., 1931	Weekly Average July, 1932	Week Aug. 6, 1932
Total private and public	$31,000	$20,500	$35,800

Cumulative, Jan. 1 to date:
1931 new capital $1,652,000
1932 new capital 553,000
Decrease, 67 per cent.

ENR Cost and Volume Index

Cost		Volume	
August, 1932	**156.80**	**July, 1932**	**166**
July, 1932	153.36	June 1932	131
August, 1931	171.38	July, 1931	248
1931 (Average)	181.35	1931 (Average)	220
1930 (Average)	202.85	1930 (Average)	260
1913 Average		100.00	

SOCIETY CALENDAR

AMERICAN ASSOCIATION OF ENGINEERS, annual convention, Washington, D. C., Sept. 26-28.

AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, annual meeting, Washington, D. C., Nov. 14-18.

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, annual meeting, White Sulphur Springs, W. Va., Oct. 11-14. (tentative).

AMERICAN PUBLIC HEALTH ASSOCIATION, annual meeting, Washington, D. C., Oct. 24-27.

AMERICAN RAILWAY BRIDGE & BUILDING ASSOCIATION, annual meeting, Toronto, Ont., Canada, Oct. 18-20.

AMERICAN SOCIETY OF CIVIL ENGINEERS, fall meeting, Atlantic City, N. J., Oct. 5-8.

AMERICAN SOCIETY OF MUNICIPAL ENGINEERS, annual meeting, New Orleans, La., Nov. 14-18.

AMERICAN WELDING SOCIETY, fall meeting, Oct. 3-7, Buffalo, N. Y.

ASPHALT INSTITUTE, Asphalt Paving Conference, New Orleans, La., Dec. 5-9. The ASSOCIATION OF ASPHALT PAVING TECHNOLOGISTS will meet in conjunction with the paving conference.

INTERNATIONAL ASSOCIATION OF PUBLIC WORKS OFFICIALS, annual conference, Pittsburgh, Pa., Sept. 29-Oct. 1.

INTERNATIONAL CITY MANAGERS ASSOCIATION and GOVERNMENTAL RESEARCH ASSOCIATION will hold a joint annual conference at Cincinnati, Ohio, on October 24 to 26.

NATIONAL ASSOCIATION OF RAILROAD AND UTILITIES COMMISSIONERS, annual meeting, Hot Springs, Ark., Nov. 15-18.

NATIONAL COUNCIL OF STATE BOARDS OF ENGINEERING EXAMINERS, annual meeting, New York City. Sept. 29-Oct. 1.

NATIONAL SAFETY COUNCIL, annual congress, Washington, D. C., Oct. 3-7.

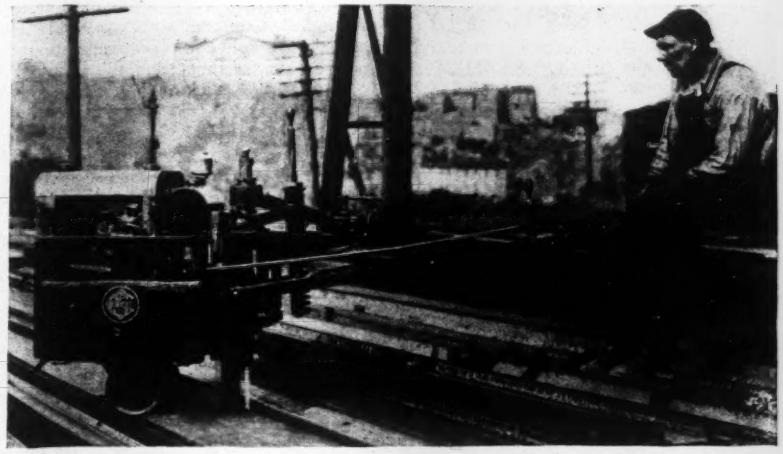
NEW ENGLAND WATER WORKS ASSOCIATION, annual meeting, Springfield, Mass., September 27-30.

NORTH DAKOTA WATER AND SEWAGE WORKS CONFERENCE will hold its fourth annual meeting at Minot, N. D., on September 26 to 28.

Construction Equipment and Materials

Gasoline Portable Drill

The Guibert Steel Co., Pittsburgh, Pa., announces its gasoline portable "Togglebug" drill. The manufacturer states that this machine will drill, ream or countersink the heaviest steel plates or sections in the field or shop. It has a variable spindle speed of 250 to 600 r.p.m. Power is furnished by a 6-hp. gasoline motor. A system of weights and counterweights gives the machine the necessary weight required for drilling large holes. The load is so balanced that the machine can be moved easily on two rollers equipped with roller bearings. When in use, the operator, by sitting or leaning on the end of the 5-ft. handle, exerts pressure on the drill point.

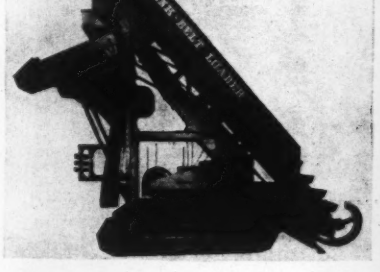


Guibert Steel Co.'s "Togglebug" drill.

Vibrating Screen Loader

The Link Belt Co., Chicago, Ill., announces a new model vibrating screen loader for coke, coal, sand, gravel, etc. A mechanically vibrated screen of the Link-Belt positively driven type is incorporated into the machine's construction. The amplitude of vibration, or throw of the screening surface, is set at the factory before the loader is shipped. The loader has a 10-ft. clearance under the chute. The bucket elevator has a capacity of 1¾ cu.yd. per minute. The foot of the loader is equipped with a helical steel ribbon feeder, which clears a path 7 ft. 7 in. wide. A three-speed transmission gives the crawlers a speed of 30 ft., or 66 ft. per minute, in the digging direction, and 27 ft. per minute in reverse.

Link-Belt vibrating screen loader.



Business Notes

AUSTIN-WESTERN ROAD MACHINERY CO., Chicago, Ill., announces that it was recently awarded an order of 56 10-ton Autocrat rollers and 69 No. 101 graders by the Pennsylvania state highway department. These machines will be used to improve a 20,000-mile primary system of township highways that are now under state control. According to the manufacturer, this is the largest single order on record for big-unit road machinery.

HARNISCHFEGER SALES CORP. has transferred L. P. Deephouse, former district manager in charge of the Montreal office, to the position of Eastern sales manager with headquarters in the Empire State Building, New York City. C. W. Daniels, formerly employed in the New York territory, has been appointed district manager in charge of the Philadelphia office.

STEPHENS-ADAMSON MFG. CO., Aurora, Ill., has established a new branch office at Rochester, N. Y., with headquarters in the Cutler Building. F. H. Wisewell has been appointed district manager and will be in charge of sales and engineering on the company's line of conveying, screening and variable-speed transmission machinery.

NOVO ENGINE CO., Lansing, Mich., announces that on July 1 Clarence E. Bement resigned as vice-president and general manager to become chairman of the board. He will be succeeded by Eric P. Teel, formerly general superintendent.

WORTHINGTON PUMP & MACHINERY CORP., with general offices at Harrison, N. J., has announced their decision to transfer and consolidate the designing, engineering and manufacturing activities formerly carried on at their Cincinnati, Ohio, works with those of their Buffalo, N. Y., manufacturing plant. For the present, the Cincinnati plant equipment will remain intact.

THE KEYSTONE DRILLER CO., Beaver Falls, Pa., announces the appointment of G. L. Harman as general sales manager. Mr. Harman was until recently sales manager of the Industrial Brownhoist Corp., Cleveland, Ohio. The Keystone Co. also announces the appointment of Lynn H. Ransom as chief engineer.

New Equipment in Brief

Cane Fiber Insulation—The Celotex Co., Chicago, Ill., announces a cane fiber building insulation that is impervious to the natural agencies of decay and to the attack of termites. The treatment given the insulating material is known as the Ferox process. This process coats each cane fiber with a chemical that is toxic to cellulose-destroying organisms. The chemical is also insoluble in water.

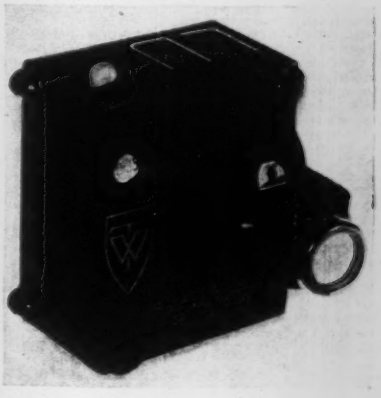
Humidity Cabinet—For providing automatic control of temperature and humidity for aging cement samples, in accordance with the requirements of the American Society for Testing Materials, the American Instrument Co., Washington, D. C., announces a humidity cabinet. The cabinet provides 21½ sq.ft. of shelf area and 13½ sq.ft. of tank area for submerged specimens. Outside dimensions are 6 ft. 8 in. long by 2 ft. 7 in. deep and 6 ft. high. Temperature is maintained by a refrigerating unit and a heating coil. A motor-driven circulating pump keeps water of a constant temperature flowing down the inside of the copper-lined walls. A relative humidity of more than 90 per cent is maintained.

Heavy-Duty Engine—The Hercules Motors Corp., Canton, Ohio, announces its IX series of heavy-duty four-cylinder power units. There are three models. The units have the same over-all dimensions and are interchangeable in connection with any assembly, the only difference being in the bore of the engines and the parts affected thereby. The three models develop 27.5, 39.5 and 46.5 hp., respectively, at 3,200 r.p.m.

Colored-Glass Comparator for Chlorination Control

An improved comparator for use in making the orthotolidin test in water-purification control, is announced by Wallace & Tiernan Co., Inc., Newark, N. J. The new comparator uses non-fading, accurately calibrated, glass color standards. The color plates are revolved in a circular disk in front of the sample, giving direct readings of residual chlorine, in parts per million, through an opening in the front of the case. The unit is inclosed in a bakelite case and is small enough to be carried in the coat pocket or bag.

Wallace & Tiernan's new comparator.



Current Construction Unit Prices

Completion of Coast Range Division, Hetch Hetchy Water Tunnel

PROVISIONS OF the new San Francisco charter required the city to call for bids for the completion of the Coast Range division of the Hetch Hetchy water tunnel. The Coast Range section, 28½ miles long, was being driven from four portals and five shafts by the Hetch Hetchy Project, a department under the Public Utilities Commission of the City and County of San Francisco, on a force account basis. At the time of taking bids for the completion of the work, there remained to be done about 6.3 miles of driving and 16 miles of lining. The city, under provisions of the new charter, was allowed to submit a bid for the work the same as outside contractors. Out of the four bids submitted for all three sections of the remainder of the work, the city bid was low, and the San Francisco Public Utilities Commission has awarded the contract to the city. This means the work will be carried on substantially as before, with the completion date set for February, 1934. The tunnel is 10 ft. 3 in. wide and 10 ft. 3 in. high at crown inside of lining, with a semicircular arch. Articles describing the tunnel, progress and methods of driving have appeared in *ENR* Jan. 15, 1931, p. 96; Jan. 31, 1929, p. 171 and Apr. 19, 1928, p. 614.

Bidders were allowed to submit proposals on one, two or three sections, or on any combination of sections. The city bid on each section and also submitted a separate proposal for all three sections, the contract being awarded on this basis for $5,257,665. Youdall Construction Co. and Geo. Pollock submitted separate bids for each section, the total being $5,811,246. Lindgren & Swinerton and Barrett & Hilp bid $5,815,335 for all three sections. MacDonald & Kahn and J. F. Shea bid $6,023,491 for the entire work. Bechtel Bros. submitted a bid of $1,258,835 for Sec. C only.

Work will go forward under the direction of M. M. O'Shaughnessy, chief consulting engineer of the commission, and Lloyd T. McAfee, chief engineer in charge of Hetch Hetchy construction.

Below is a tabulation of unit prices for all bids received except that of Bechtel Bros.

Item and Unit	Quantities				City Forces (Contract)				Youdall Construction Co. and Geo. Pollock			Lindgren, Swinerton Barrett & Hilp	MacDonald & Kahn and J. F. Shea,
	Sec. A	Sec. B	Sec. C	Total	Sec. A	Sec. B	Sec. C	Total	Sec. A	Sec. B	Sec. C	Secs. A-B-C	Secs. A-B-C
Tunnel and 10-in. lining, lin.ft.	600	400	200	1,200	$82.00	$82.00	$80.00	$80.00	$67.50	$80.50	$78.00	$149.00	$80.00
Tunnel and 14-in. lining, lin.ft.	900	400	100	1,400	101.00	104.00	98.00	99.00	84.50	97.00	97.50	150.00	110.00
Tunnel and 16-in. lining, lin.ft.	2,700	700	200	3,600	108.00	111.00	105.00	106.00	113.20	130.00	130.70	154.00	120.00
Tunnel and 18-in. lining, lin.ft.	3,000	1,200	200	4,400	116.00	119.00	113.00	114.00	133.50	146.50	156.00	157.00	135.00
Tunnel and 21-in. lining, lin.ft.	4,300	3,400	700	8,400	126.00	129.00	123.00	124.00	152.10	170.50	174.00	159.00	143.00
Tunnel and 24-in. lining, lin.ft.	500	300	100	900	137.00	140.00	134.00	134.00	189.00	200.00	215.00	169.00	170.00
Tunnel and 28-in. lining, lin.ft.	200	100	100	400	150.00	153.00	147.00	146.00	220.00	240.00	254.00	171.00	225.00
Tunnel and 21-in. lining between Alameda Ck. east hdg. and Indian Ck. west hdg., lin.ft.	.....	.....	100	100	.....	.....	139.00	134.00	.....	.....	425.00	700.00	500.00
Widen drift prev. excav., cu.yd.	.....	.....	600	600	.....	.....	15.00	15.00	.....	.....	13.00	9.00	13.00
Excav. bench under existing drift, cu.yd.	.....	.....	5,000	5,000	.....	.....	4.00	4.00	.....	.....	10.00	3.00	8.00
Timbering two prev. items, M.b.m.	.....	.....	200	200	.....	.....	60.00	60.00	.....	.....	50.00	40.00	100.00
Enlarge tunnel already excav., cu.yd.	300	400	300	1,000	12.00	12.00	12.00	12.00	4.40	4.40	4.40	9.50	24.00
Excavate invert in tunnel already driven, cu.yd.	3,000	5,000	2,000	10,000	6.00	6.00	6.00	6.00	3.41	3.41	3.30	6.50	6.00
Excavation, overflow shaft, cu.yd.	.....	.....	120	120	.....	.....	5.00	5.00	.....	.....	15.00	12.00	12.00
Fill lower corners tunnel with material from outside, cu.yd.	200	700	300	1,200	1.25	1.25	1.25	1.25	2.50	2.50	2.50	3.00	5.00
Retimbering with new timb., M.b.m.	40	40	20	100	60.00	60.00	60.00	60.00	50.00	50.00	50.00	90.00	100.00
Retimbering with used timb., M.b.m.	80	80	40	200	35.00	35.00	35.00	35.00	25.00	25.00	25.00	90.00	75.00
Readjust timbering, M.b.m.	150	250	100	500	10.00	10.00	10.00	10.00	14.00	14.00	14.00	42.00	50.00
Removal of timbering, M.b.m.	2,400	3,300	2,300	8,000	15.00	15.00	15.00	15.00	7.50	7.50	7.50	2.00	10.00
Additional conc. in new tunnel, cu.yd.	300	300	200	800	8.95	8.95	8.45	8.70	6.60	6.30	6.00	10.00	12.00
Concrete lining in previously excav. tunnel, cu.yd.	46,000	83,000	41,000	170,000	8.95	8.95	8.45	8.70	7.80	8.475	9.00	8.00	10.00
Completing lining where invert is already in place, cu.yd.	.....	.....	31,000	31,000	.....	.....	7.70	7.70	.....	.....	8.90	8.00	10.00
Gunite lining in advance at regular lining, cu.yd.	600	200	200	1,000	23.00	23.00	22.00	22.00	23.00	23.00	23.00	28.00	20.00
Poured lining in advance at regular lining, cu.yd.	300	300	100	700	17.00	17.00	16.00	16.00	12.00	12.00	12.00	30.00	23.00
Surface finish on gunite lining already partially completed, sq.ft.	64,000	66,000	119,000	249,000	.25	.25	.25	.25	.30	.28	.28	.25	.35
Concrete in shafts, cu.yd.	1,050	1,850	100	3,000	21.00	21.00	20.00	20.00	15.00	15.50	14.00	14.00	15.00
Conc., underground, other than in tunnels and shafts, cu.yd.	300	500	150	950	26.00	26.00	25.00	25.00	9.50	9.50	9.00	10.00	15.00
Gunite, underground, other than tunnels and shafts, cu.yd.	100	300	300	700	28.00	28.00	27.00	27.00	21.00	21.00	20.50	25.00	18.00
Drilling ¾-in. test holes, lin.ft.	1,000	1,200	1,000	3,200	.25	.25	.25	.25	.44	.44	.44	.75	1.00
Drilling grout holes, lin.ft.	1,000	1,200	1,000	3,200	.25	.25	.25	.25	.66	.66	.66	1.00	1.50
Furnish and place grout pipes, lin.ft.	3,200	3,900	3,200	10,300	.25	.25	.25	.25	.55	.57	.55	.30	1.00
Making con. to grout pipes, each	1,500	1,800	1,500	4,800	2.00	2.00	2.00	2.00	6.27	6.27	6.27	.25	1.00
Furnish sand or grout, tons	14,000	14,000	17,000	45,000	2.90	2.60	2.10	2.50	2.10	1.85	1.82	1.45	1.75
Pressure grouting, cu.yd.	14,000	14,000	17,000	45,000	3.60	3.60	3.60	3.60	6.50	6.63	7.75	3.30	6.00
High pressure grouting, cu.yd.	100	100	50	250	6.00	6.00	6.00	6.00	6.60	6.60	6.60	7.00	20.00
Earth excav., open cut, cu.yd.	500	100	700	1,300	1.00	1.00	1.00	1.00	1.30	1.25	1.25	1.50	1.00
Rock excav., open cut, cu.yd.	100	50	100	250	3.00	3.00	3.00	3.00	2.00	2.00	2.00	3.00	2.50
Refill. and embanking, cu.yd.	200	100	400	700	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	.75
Concrete above ground, cu.yd.	100	50	275	425	12.00	12.00	12.00	12.00	25.20	25.20	25.20	30.00	13.00
Gunite lining in canals, sq.ft.	8,200	.....	.....	8,200	.25	.....	.....	.25	.22	.....	.....	.25	.18
Reinf. steel in tunnel lining, lb.	300,000	300,000	300,000	900,000	.08	.08	.08	.08	.045	.045	.042	.04	.05
Reinf. steel underground except otherwise covered, lb.	130,000	300,000	70,000	500,000	.08	.08	.08	.08	.047	.0475	.046	.04	.05
Reinf. steel above ground, lb.	9,000	1,000	35,000	45,000	.05	.05	.05	.05	.055	.055	.052	.04	.05
Metal drip pans or shields, sq.ft.	3,500	6,500	7,500	17,500	.10	.10	.10	.10	.07	.07	.07	.15	.20
Misc. steel castings, lb.	11,000	11,000	6,000	28,000	.20	.20	.20	.20	.15	.15	.15	.10	.15
Misc. structural steel, lb.	80,000	80,000	35,000	195,000	.15	.15	.15	.15	.17	.17	.17	.07	.10
Galvanizing, lb.	40,000	40,000	20,000	100,000	.02	.02	.02	.02	.02	.02	.02	.02	.05
C.I. pipe and fittings, lb.	1,600	1,600	800	4,000	.10	.10	.10	.10	.09	.09	.09	.10	.15
Misc. bronze, brass, copper, lb.	1,300	1,200	700	3,200	.40	.40	.40	.40	.50	.50	.50	.40	.50
Misc. lead, lb.	500	500	1,000	2,000	.15	.15	.15	.15	.06	.06	.06	.20	.20
Placing metal work furn. by city, lb.	40,000	5,000	5,000	50,000	.05	.05	.05	.05	.05	.05	.05	.05	.05
Extra cement for all concrete and for grouting, bbl.	40,000	51,500	38,500	130,000	2.15	2.15	2.00	2.10	2.00	2.00	2.00	2.00	1.95
Total					$2,205,590	$1,984,575	$1,152,525	$5,257,665	$2,327,433	$2,110,046	$1,373,767	$5,815,335	$6,023,491